CHAPTER 9.0
CONSTRUCTION MONITORING AND QUALITY ASSURANCE

The successful transfer of design objectives into construction is accomplished by consideration of construction operations during the design phase. In recent years the amount of coordination between design and construction has steadily decreased; primarily due to graduate engineers who specialize in design and who are never exposed to construction operations. In past years, engineers either began their careers in construction and advanced into design, or were assigned the design and construction responsibilities for projects. Present lack of coordination stemming from inexperience with field operations can result in a technically superior set of construction plans and specifications, which cannot be built. Rational construction control is vital to assure a safe, cost-effective foundation and to avoid unnecessary court of claims actions.

9.1 EARTHWORK AND SPREAD FOOTING FOUNDATIONS

Approach embankment construction should be clearly defined in standard drawings as to materials and limits of placement. Such standards assure uniformity in construction due to the familiarity of the construction personnel with the operations being performed and results expected. The designer should keep to the standard unless major changes are required. Attempts at small changes in materials or limits are counterproductive to good construction.

The philosophy of approach embankment compaction is to insure adequate bearing capacity for abutments (or piers) placed in the embankment and to minimize settlement of the pavement or footing. Typical highway embankments require compaction to 90 percent of maximum density (AASHTO T180) to control pavement settlement. Designers of approach embankments should specify 95 percent of T180 to limit differential settlement between the structure and fill. If piles are used to support footings in fill, the maximum top size of embankment material should be limited to 6 inches to ease pile installation. If spread footings are used, a minimum of 5 feet of select material compacted to 100 percent of T99 should be placed beneath the footing and extended to beyond the wingwalls. This layer provides uniform support for the footing and a rigid transition between the structure-fill interface to minimize differential settlement. Construction control is usually keyed to percent compaction on the standard design drawings.

Construction of spread footings on soil must be controlled such that a stable surface exists on which to pour the footing. The designer should anticipate situations where construction operations may temporarily disturb the foundation soil, i.e., footing elevation in fine sand near the water table. A note should be included to alert the engineer of the potential problem and what action to take, i.e., if unstable soil is encountered at footing level, undercut one foot and backfill with gravel to footing level.

Assurance of the footing being placed on the proper soil can be guaranteed by including a soil profile in the contract plans and requiring inspection of the prepared footing level by either a representative of the geotechnical engineer or a construction inspector who can confidently verify actual foundation conditions.

Spread footing excavations frequently require sheeting to retain the excavation walls while the footing is poured. In cases where sheeting extends below footing level within three feet of the footing sides, consideration should be given to leaving the sheeting in place if:

1. Uniform footing settlement magnitude is critical, i.e., less than 1/2 inch.

2. Sheet pile Z - sections are used (because a large quantity of soil may remain stuck between adjacent flanges and be removed with sheeting).

3. Sheeting will only be pulled on one side (can cause differential settlement).

9.2 EMBANKMENT CONSTRUCTION MONITORING - INSTRUMENTATION

The observational approach to design involves monitoring subsoil behavior during early construction stages to predict responses to subsequent construction. Basic soil mechanics concepts can be used to accurately predict future subsoil behavior if data from instrumentation is analyzed after initial construction loads have been placed. Occasionally a design problem arises which is unique or of major criticality that can only be safely solved by utilizing the observational approach.

Embankment placement must be carefully observed and monitored on projects where stability and/or settlement are critical. The monitoring should include visual observation by the construction inspection staff and use of instrumentation. Without the aid of various forms of instrumentation, it is impossible to determine what is happening to the foundation. Instrumentation can be used to warn of imminent failure or to indicate whether settlement is occurring as predicted. The type of instruments to be used and where they will be placed should be planned by a qualified soils engineer. Actual interpretation and analysis of the data from the instrumentation should also be done by someone with a background in soil mechanics; however, the project engineer and inspector should understand the purpose of each type of instrumentation and what the data is to be used for.

9.2.1 Inspector's Visual Observation

In areas of marginal embankment stability, the inspector should walk the surface of the embankment daily looking for any sign of cracking or movement. Hairline cracks often develop at the embankment surface just prior to failure. If the inspector should discover any such indication, all fill operations should cease immediately. All instrumentation should immediately be read. The soils engineer should be notified. Subsequent readings will indicate when it is safe to resume operations. Unloading by removal of fill material is sometimes necessary to prevent an embankment failure.

9.2.2 Types of Instrumentation

The usual instrumentation specified to monitor foundation performance on projects where stability and settlement are critical consists of:

1. Slope Inclinometers are used to monitor embankment stability. A slope inclinometer consists of a 2 to 3-inch grooved plastic or metal tube that is installed in a borehole. The bottom of the slope inclinometer tube must be founded in firm soil or rock. A readout probe is lowered down the tube and deflection of the tube is measured. With a slope inclinometer, the amount and location of horizontal movement in the foundation soil can be measured. For embankments built over very soft subsoils, telescoping inclinometer casing should be used to account for vertical consolidation. In soft ground conditions, several inches of lateral movement (squeeze) may occur without shear failure as the embankment is built. Therefore, from a practical construction control standpoint, the rate of movement rather than the amount is the better indicator of imminent failure. Slope

inclinometer readings should be made often during the critical embankment placement period (daily if fill placement is proceeding rapidly) and readings should be plotted immediately on a time versus movement plot. Fill operations should cease if a sudden increase in rate of movement occurs.

2. Piezometers indicate the amount of pressure build-up within the water-saturated pores of the soil. There are critical levels to which the water pressure in the subsoil will increase just prior to failure. The soils engineer can estimate the critical water pressure level during design. Normally, the primary function of piezometers during fill placement is to warn of failures. Once the embankment placement is complete, the piezometers are used to measure the rate of consolidation. There are several different types of piezometers. The simplest is the open-standpipe type, which is essentially a well point with a metal or plastic pipe attached to it. The pipe is extended up through the fill in sections as the fill height increases. This type has the disadvantage that the pipes are susceptible to damage if hit by fill construction equipment. There are several types of remote piezometers that eliminate the requirement for extending a pipe up through the fill. Also the response time of open well piezometers is often too slow in soft clays to warn of potential embankment failure. The remote units consist of a piezometer transducer that is sealed in a borehole with leads carried out laterally under the base of the embankment to a readout device, which measures the porewater pressure. Pneumatic or vibrating wire piezometers have rapid response to changes in pore pressure.

3. Settlement devices are used to measure the amount and rate of settlement of the foundation soil due to the weight of the embankment. They are installed on or just below the existing ground surface before any fill is placed. The simplest settlement device is a settlement platform (usually a 3 or 4 foot square plywood mat or steel plate) with a vertical reference rod (usually 3/4-inch pipe) attached to the platform. The reference rods are normally added 4 feet at a time as the height of the embankment increases. The elevation of the top of the reference rod is surveyed periodically to measure the foundation settlement. Remote pneumatic settlement devices are also available. As with the remote piezometer devices, the remote settlement devices have the advantage of not having to bring a reference rod up through the fill.

9.2.3 Typical Instrument Locations

Instrument installations should be spaced approximately 250-500 feet along the roadway alignment in critical areas. Typical instrument locations for an embankment over soft ground are shown in Figure 9-1:

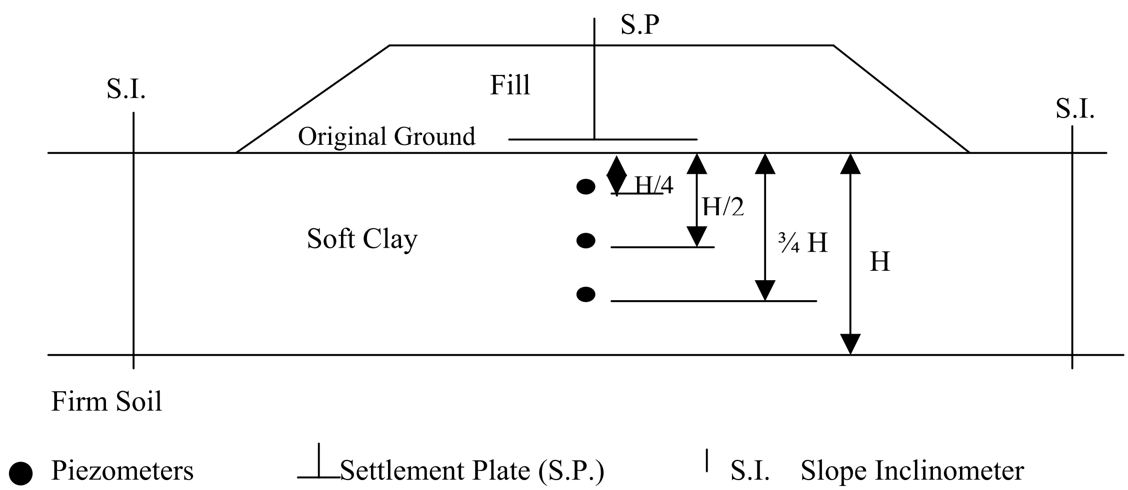

Figure 9-1: Typical instrumentation location for an embankment over soft ground

9.3 PILE FOUNDATIONS

Construction control of pile operations is a much more difficult proposition than for spread footings. During footing placement an inspector can easily examine a prepared footing area and watch the concrete footing being poured to assure a quality foundation. Piles derive their support below ground. Direct quality control of the finished product is not possible. Therefore, substantial control must be maintained over the peripheral operations leading to the incorporation of the pile into the foundation. In general terms, control is exercised in three areas; the pile material, the installation equipment, and the subsurface soil resistance. These items are interrelated with changes in one affecting the others. It is mandatory that pile foundation installation be considered during design to insure that the piles shown on the plans can be installed.

9.3.1 SELECTION OF DESIGN SAFETY FACTOR BASED ON CONSTRUCTION CONTROL

The safety factor to be selected for a pile foundation depends on both design aspects and construction control techniques. End bearing piles may be designed with a safety factor of 2.0 if adequate control is exercised in pile section design and hammer approval. Assuming that an adequate foundation investigation and static analysis has been performed for a friction pile foundation, the following safety factors are generally recommended to be applied to ultimate capacities of piles supported by soil based on construction control techniques to be used.

Construction Control Technique	Global Safety Factor
Representative static load test	2.00
Representative dynamic load test	2.25
Wave equation analysis and indicator piles	2.50
Wave equation	2.75
Dynamic formula	3.50

Piles supported primarily by end bearing on rock with RQD > 50 may use a safety factor of 2.0 assuming driveability has been confirmed by wave equation analysis. End bearing piles on poorer rock or intermediate geomaterials should use safety factors which consider both construction control and the previous history of end bearing piles in that particular formation.

9.3.2 RESPONSIBILTY FOR QUALITY ASSURANCE

Clear lines of responsibility are needed to permit successful installation of pile foundations. The designer is generally responsible for selection and display on the plans of the following information:

1. Pile details

    a. Material - concrete, steel or timber

    b. Allowable stresses - design and driving

    c. Cross section - diameter, tapered or straight, and wall thickness

d. Estimated length

e. Pile design load

2. Soils data

a. Subsurface profile

b. Soil resistance to be overcome to reach estimated length (as determined by static pile analysis).

c. Special notes - boulders, artesian pressure, buried obstructions, etc.

3. Pile Installation

a. Method of hammer approval

b. Special notes - spudding, preaugering, jetting, reinforced tips, etc.

c. Estimated blow count at estimated length

The engineer on construction is generally responsible for quality control of the following items and for initiating communication with the designer on variations from the plans.

1. Pile

a. Quality control testing or certification of materials.

2. Soils data

a. Major discrepancies in soil profile reported to designer as soon as encountered.

3. Installation

a. Hammer maintained in good working order - cushion replaced regularly.

b. Final pile length determined from estimated blow count, estimated length and subsurface profile.

c. Pile stress controlled.

d. Documentation of field operations.

Proper construction control of pile driving requires good communication between designer and field engineer. Such communication cannot follow traditional lines and still be effective. Answers are needed in a short time to prevent expensive contractor down time or to prevent pile driving from continuing in an unacceptable fashion.

Good communication should begin with a preconstruction meeting of the foundation designer and field engineer. The designer should briefly explain the design and point out possible problem areas. The most important purpose is to establish a direct line of communication between the designer and the field. The designer should advise the field engineer, on request, of the design aspects of problems occurring in construction. The ultimate decision on any field problem must remain in the traditional lines of authority established for construction. Interaction between office and field will simplify and expedite decisions.

9.4 PILE DRIVING EQUIPMENT AND OPERATION

Proper inspection of pile driving operations requires that the inspector have a basic understanding of pile driving equipment. Estimation of "as driven pile capacity" is usually based on the number of hammer blows needed to advance the pile a given distance. Each hammer blow transmits a given amount of energy to the pile. The total number of blows is the total energy required to move the pile a given distance. This energy can then be related to soil resistance and supporting capacity. However, pile inspection entails more than counting blows of the hammer.

The energy transmitted to the pile by a given hammer can vary greatly depending on the equipment used by the contractor. Energy losses can occur by poor alignment of the driving system, improper or excessive cushion material, improper appurtenances or a host of other reasons. As the energy losses increase, more blows are required to move the pile. The manufacturer's rated hammer energy is based on minimal energy losses. Assumptions that the hammer is delivering its rated energy to the pile can prove dangerous if substantial energy is lost in the driving system. Artificially high blow counts can result in acceptance of driven pile lengths, which are shorter than that necessary for the required pile capacity.

Important elements in the driving system include the leads, the hammer cushion, the helmet, and for concrete piles, the pile cushion. Typical components of a pile driving system are shown in Figure 9-2. The leads are used to align the hammer and the pile such that every hammer blow is delivered concentrically to the pile system. The helmet holds the top of the pile in proper alignment and prevents rotation of the pile during driving. Typical components of a helmet are shown in Figure 9-3. Both the hammer and the helmet ride in the leads so that hammer - pile alignment is assured. Inspectors should be concerned about "flying leads" which sit on top of the pile and only control the hammer alignment. Erratic energy delivery to the pile can be caused by misalignment of such lead systems

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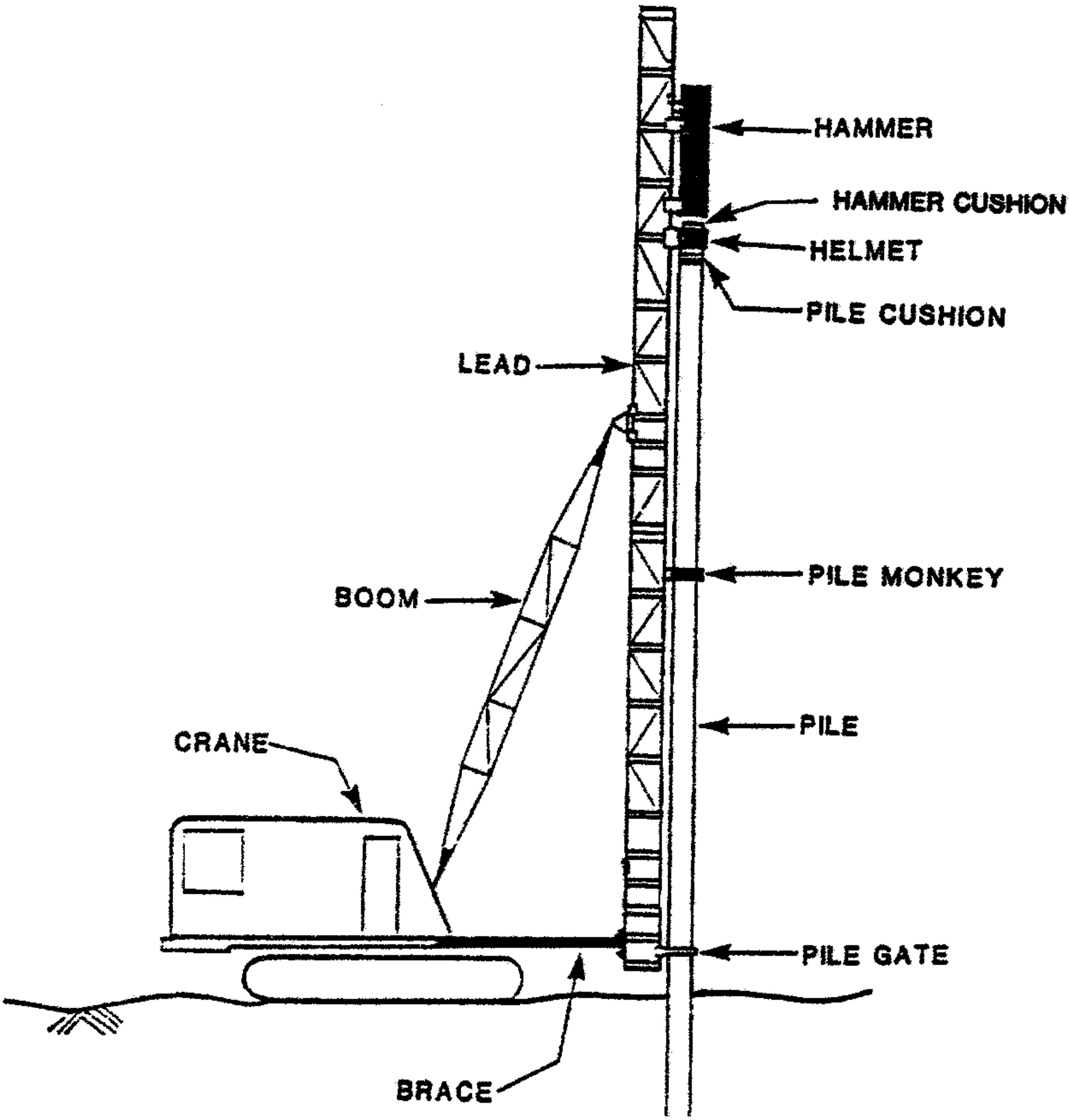
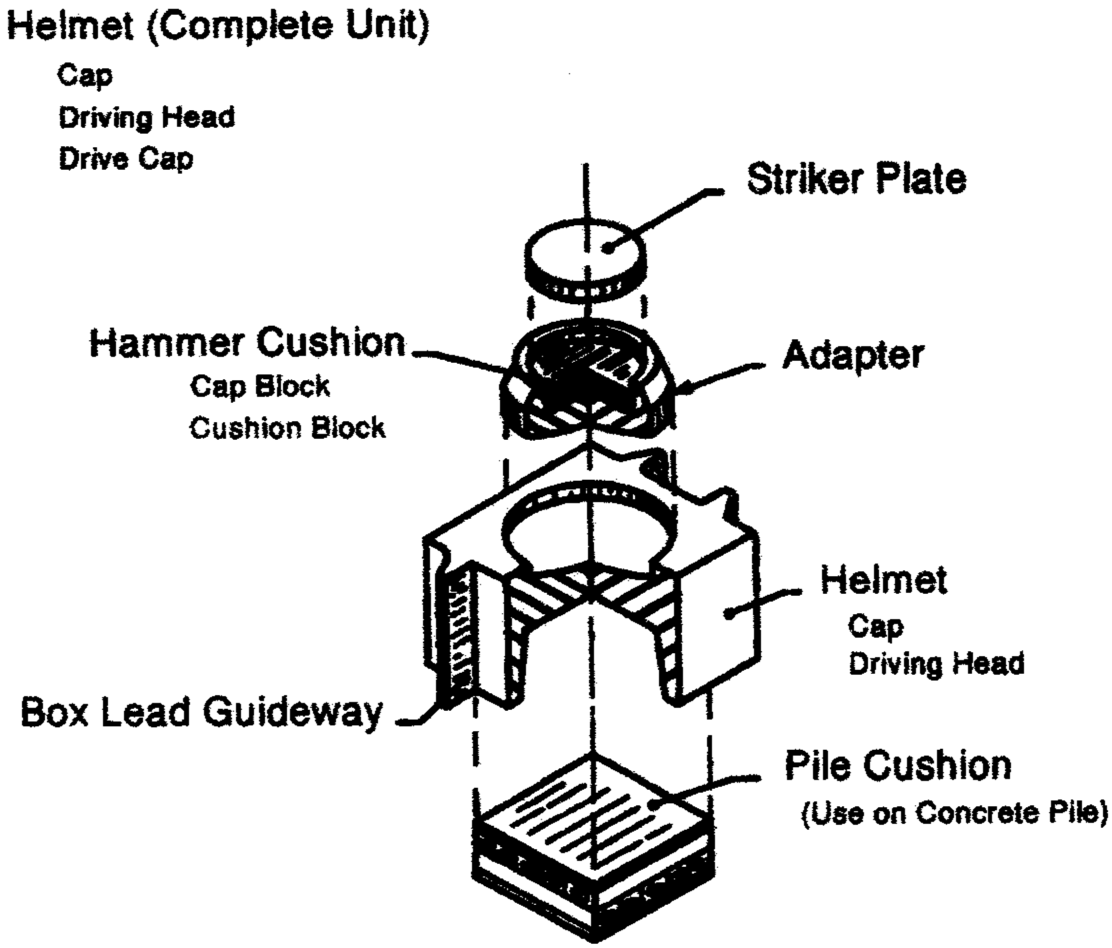


Pile Support System

Figure 9-2: Typical components of a pile driving system



Note: The helmet shown is for nomenclature only. Various sizes and types are available to drive H, pipe, concrete (shown) and timber piles. A system of inserts or adapters is utilized up inside of the helmet to change from size to size and shape to shape.

Figure 9-3: Typical components of a helmet

All impact pile driving equipment, except some gravity hammers shall be equipped with a suitable thickness of hammer cushion material. The function of this material is to prevent damage to the hammer or pile and insure uniform energy delivery per blow to the pile. Hammer cushions shall be made of durable manufactured materials provided in accordance with the hammer manufacturer's guidelines, except that all wood, wire rope and asbestos hammer cushions are specifically disallowed and shall not be used. The thicker the hammer cushion, the less the energy that is transferred to the pile. Mandatory use of a durable hammer cushion material, which will retain uniform properties during driving, is necessary to accurately relate blow count to pile capacity. Non-durable materials, which deteriorate during driving cause erratic energy delivery to the pile and prevent the use of blow counts to determine pile capacity.

The heads of concrete piles shall be protected by a pile cushion made of plywood. The minimum thickness of pile cushion placed on the pile head shall not be less than four inches. A new pile cushion should be provided for each pile.

A non-routine element called a follower may be used in the driving system, particularly for piles driven below water. Followers cause substantial and erratic reduction in the hammer energy transmitted to the pile due to the follower flexibility, poor connection to the pile head, frequent misalignment, etc. Reliable correlation of blow count with pile capacity is impossible when followers are used. Special monitoring with devices such as the pile analyzer is required when followers are used.

9.5 DYNAMIC PILE DRIVING FORMULAE

In the 1800's the fundamental pile driving formula was established to relate dynamic driving forces to available pile bearing capacity. The formula was based on a simple energy balance between the kinetic energy of the ram at impact and the resulting work done on the soil, i.e., a distance of pile penetration against a soil resistance. The concept assumed a pure Newtonian impact with no energy loss. The fundamental formula was expressed as follows:

KINETIC ENERGY INPUT = WORK DONE ON SOIL

$$\therefore WH = RS \tag{9-1}$$

Where:
- W = Weight of the ram
- H = Distance of ram fall
- R = Total soil resistance (driving capacity) against the pile
- S = Pile penetration (set) per blow

Using this simple energy approach, the total soil resistance (driving capacity) could be calculated as:

$$R = \frac{WH}{S} \tag{9-2}$$

An inherent difficulty in the pile driving operation is that a small portion of the ram's kinetic energy actually causes penetration of the pile. Studies indicate that typically only 30 to 65 percent of the rated energy is passed thru to the pile. Much energy is lost in either heat (soil friction, hammer mechanism, pile material, etc.) or strain (elastic compression of the cushion, the pile and the surrounding soil. An attempt was made to quantify these losses in the late 1800's by observing pile driving operations. The result was the ENR pile driving formula which related the safe load that a pile could withstand to the input energy and set per blow. Basically the equation was developed to lump all losses and safety factors into a single factor. The formula follows:

$$P = \frac{2WH}{S + k} \tag{9-3}$$

Where:
- P = Safe pile load in kips
- W = Weight of ram in kips
- H = Distance of ram fall in feet
- S = Set per blow in inches
- k = Constant which varies from 0.1 to 1 based on hammer type

Observe that the ENR formula is not dimensionally correct as H is in feet and S is in inches. The overall safety factor can be roughly determined by comparing the ENR formula for safe load to the fundamental formula (which is for total driving capacity) as follows:

1. Change ENR to dimensionally correct form by changing H from feet to inches,

$$P = \frac{2WH' \times 12}{(S'' + k) \times 12} \qquad \text{i.e., H (feet) x 12 = H'' (inches)}$$

$$P = \frac{2WH''}{12(S'' + k)}$$

$$P \cong \frac{WH''}{6S''} \qquad \text{(assumes k is small)} \qquad (9\text{-}4)$$

2. Compare to total driving capacity,

$$R = \frac{WH}{S}$$

3. Safety Factor = $\frac{R}{P} \cong 6$

Most engineers are not aware of (1) the use for which the ENR formula was originally developed, or (2) the fact that the ENR formula has a built-in factor of safety of 6. Sowers, in his 1979 Introductory Soil Mechanics and Foundation Engineering Text, states the following about the ENR formula: "The ENR formula was derived from observations of the driving of wood piles in sand with free-falling drop hammers. Numerous pile load tests show that the real factor of safety of the formula can be as low as 2/3 and as high as 20. For wood piles driven with free-falling drop hammers and for lightly loaded short piles driven with a steam hammer, the ENR formulas give a crude indication of pile capacity. For other conditions they can be very misleading."

In 1988 the Washington State DOT (WDOT) published a study entitled "Comparison of Methods for Estimating Pile Capacity," Report No. WA-RD-163.1. The study which was based on high quality pile load test data, showed the ENR formula to be the least reliable of the 10 dynamic formulae which were analyzed. More recent studies by FHWA under Demonstration Project 66 have also confirmed the unreliability of the ENR formula, particularly for higher pile loads where actual safety factors are too frequently less than 1.0.

The WDOT study and the FHWA Demonstration Project 66 resulted in both organizations replacing ENR in their specifications with the Gates dynamic formula. However, the formula is usually restricted to piles which have driving capacities less than 600 kips. The Gates formula was originally developed based on correlations with static load test data. The Gates formula, which was modified by FHWA for driving capacity, is shown below:

$$R = 1.75\sqrt{E}\ \mathrm{Log}(10N) - 100 \qquad (9\text{-}5)$$

Where: R = Driving Capacity (kips)
E = Manufacturer rated energy (foot-pounds) at the stroke observed in the field

Log (10N) = Logarithm to the base 10 of the quantity 10 multiplied by N, the number of hammer blows per inch at final penetration (blows per inch)

9.6 DYNAMIC ANALYSIS OF PILE DRIVING

An examination of the pile driving process discloses that the concept of a Newtonian impact does not apply. When viewed in slow motion, the ram does not immediately rebound from the pile after impact. The ram transfers force to the pile head over a finite period of time which depends on the properties of the hammer-pile-soil system. A force pulse is created which travels down the pile in a wave shape. The amplitude of the wave will decay due to system damping properties before reaching the pile tip. The force in the wave, which reaches the tip, will "pull" the pile tip into the soil before the wave is reflected back up the pile. After reflection an amount of permanent "set" of the pile tip will remain. This process is crudely shown in (Figure 9-4) for the hammer-pile-soil system.

Each element in the hammer-pile-soil system affects the pile penetration and stresses caused in the pile. A few characteristic effects of each element are discussed below.

1. Hammer

   - Mechanical efficiency determines what percentage of rated energy is transmitted by the ram. Typical values of percent energy transfer for hammers in good repair are 50% for air-steam, 70% for diesel and over 90% for hydraulic.

   - Force wave shape characteristics are different for different hammer types. The shape affects pile stress and pile penetration. Air-steam types generate a wave with high amplitude and a low period. Equivalent diesel hammers generate lower amplitudes but longer periods.

2. Pile and Appurtenances (Cushions, Helmets, Etc.)

   - The stiffness of appurtenances such as the hammer cushion is defined by the cross sectional area times the modulus of elasticity divided by the thickness. The stiffness has a major effect on both blow count and stress transfer to the pile. These elements must not degrade during driving as observed blow count will decrease and pile stresses increase.

   - The cross sectional area of the pile is a major factor in pile driveability. Long piles with small cross sectional areas are so flexible that the hammer energy is absorbed in strain rather than as work to advance the pile against the soil resistance. Pile aids such as mandrels are used to temporarily increase the pile cross section (and stiffness) during driving.

3. Soil

   - Soil strength may be permanently or temporarily changed during driving. Piles being driven into soil containing large percentages of fines may require restrikes to estimate long term capacity due to effects of set-up or relaxation.

   - The damping properties of the soil surrounding the pile can have dramatic effect on observed blow count. An increase in damping decreases driveability. Damping parameters can be estimated by soil type or from basic index test data. Consideration of the dynamic aspects of the field pile driving operation is necessary to relate to the static pile capacity. Foundation designers should routinely consider the potential for dynamic effects such as set-up and include provisions for field observations such as restrikes. In addition, construction control of pile driving should account for basic dynamic parameters which influence blow count and pile stress. Some can be controlled by specification; others require use of a pile wave equation analysis.

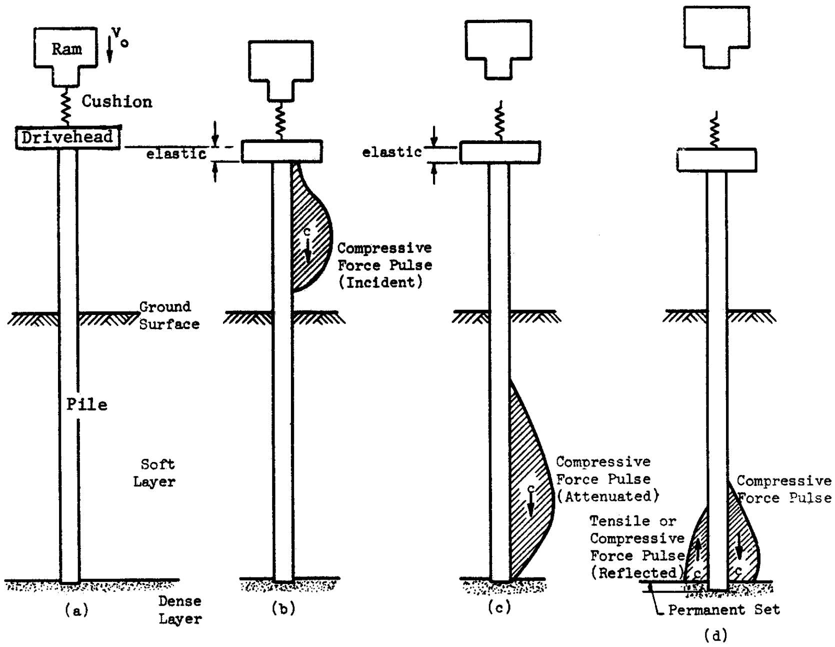


Figure 9-4: Hammer-pile-soil system

9.7 WAVE EQUATION ANALYSIS

The wave equation analysis is a computer-based analysis developed from the one dimensional wave equation. The classical wave theory was developed to model wave propagation in a slender rod subjected to an applied force at one end. The pile wave equation uses the wave propagation theory to define the longitudinal wave caused by a hammer impact at the pile top. However, the classical theory must be modified to account for changes in the traveling wave form due to pile and soil properties. Currently the model on which wave equation analysis is based contains a series of masses interconnected by a system of springs and dashpots. These latter elements attenuate the traveling force wave. Pile set is caused by the portion of the force wave which reaches the pile tip. If the traveling wave is completely dissipated by the pile and soil properties prior to reaching the tip, no permanent set will occur and the pile head will rebound. Wave dissipation commonly occurs on projects where either too small a hammer is used or too small a pile cross sectional area is specified for the length being driven.

1. Input to Pile Wave Equation

   Input parameters are required for the hammer, pile (plus appurtenances), and soil. The confidence level which can be assigned to the output is directly related to how well the input parameters are known. The basic input parameters are discussed below.

   Hammer input properties are usually well known and stored in a data file in the wave equation program. In design analysis, hammer types are selected based on the soil resistance to be overcome.

In construction control analysis the contractor submits the hammer type. The major concern in construction is that the hammer is in good working condition as was assumed for the input.

Appurtenance input consists of weight and/or stiffness values. The properties of cushions are especially critical. Only manufactured materials whose properties remain constant during driving can be used with confidence. The actual cushion thickness used in the field must be checked and discrepancies reported so that pile wave analysis can be modified.

Pile length and cross sectional area are major input items. The pile wave analysis cannot predict pile length. This fact is commonly misunderstood by engineers. Pile length is determined by static analysis procedures and then used as input to pile wave analyses. Cross sectional area of the pile is frequently varied in design analyses to determine which section is both driveable and cost effective. Increasing the pile section has the effect of improving driveability as well as reducing pile stresses.

Soil data input requires both an understanding of site specific soil properties and the effects of pile driving on those properties. Dynamic properties such as damping and quake are roughly correlated with soil type. These properties are best determined by experienced geotechnical engineers. The driving soil resistance and its distribution are determined from the static analysis. Remember that the driving soil resistance may be substantially greater than the design load times the safety factor; particularly for scour piles. Also the dynamic effects of pile driving on soil resistance must be considered by an experienced geotechnical engineer to determine set-up or relaxation values for ultimate soil resistance. These dynamic effects are frequently overlooked and can result in large variations between estimated and actual pile lengths.

2. Output Values from Pile Wave Analysis

The results of a pile wave analysis include the predicted blow count, pile stresses, and delivered hammer energy for an assigned driving soil resistance, R_{ult}, assuming given hammer, appurtenance, and pile conditions. Remember that each pile wave analysis is for the specific pile length that was input. The summary table of output shown below was generated for a specific site where a pile length of 50 feet was being analyzed.

WAVE EQUATION SUMMARY

R_{ult} kips	Blow Count BPF	Stroke (EQ) Ft.	Tensile Stress Ksi	Compressive Stress Ksi	Transfer Energy Ft-Kip
35.0	7	3.27	-0.73	1.68	13.6
80.0	16	3.27	-0.32	1.71	13.6
140.0	30	3.27	-0.20	1.73	13.0
160.0	35	3.27	-0.14	1.73	13.0
195.0	49	3.27	-0.00	1.75	12.8
225.0	63	3.27	0.0	1.96	12.7
280.0	119	3.27	0.0	2.34	12.6
350.0	841	3.27	0.0	2.75	12.5

** Note that for each driving resistance (R_{ult}), a value of blow count, hammer stroke, tensile stress, compressive stress, and transferred energy has been computed. The data is also commonly shown in graphical form as noted in the following plot.

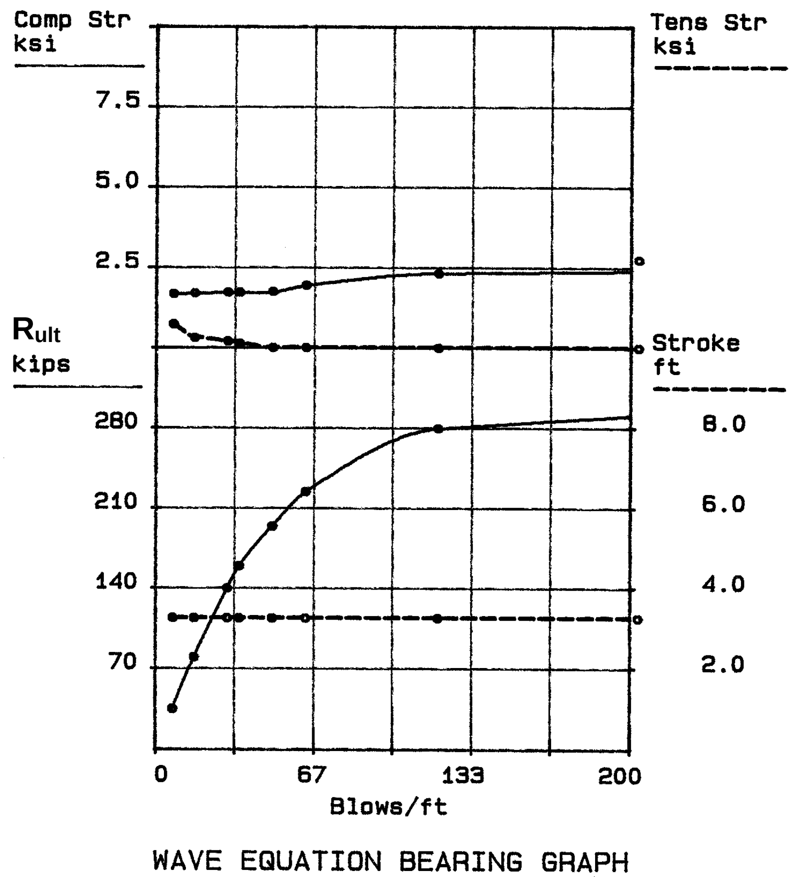


Figure 9-5: Grlweap – Summary of Compressive Stress, Tensile Stress, and Driving Capacity vs. Blow Count

3. Pile Wave Equation Analysis Interpretation

The summary table of output shown in Figure 9-5 contains the predicted relationship between pile hammer blow count and other variables for the situation when the pile is embedded 50 feet in the ground. Therefore, the data in the table is interpreted in the field by comparing the measured blow count at a pile penetration of 50 feet with the data in the summary table i.e., when the pile reaches 50 feet, if the blow count is 49, the driving capacity is 195 kips, the stroke 3.27', the tensile stress zero ksi, the compressive stress 1.75 ksi, and transferred energy 12.8 ft-kips. If the blow count had been 63 the driving capacity would have been predicted to be 225 kips, etc.

Note that this summary table is for an air-steam hammer and the stroke is constant for all blow counts. Diesel hammers operate at different strokes depending on the pile-soil properties. A pile wave summary table for a diesel hammer will display a predicted combination of blow count and stroke which is necessary to achieve the driving capacity. In fact, there are numerous combinations of blow count and stroke which correspond to a particular driving capacity. These combinations may be computed and plotted for a selected driving capacity using the constant capacity output option of the wave equation. A typical plot of diesel hammer stroke versus blow count is shown in Figure 9-6 for a constant capacity of 240 kips.

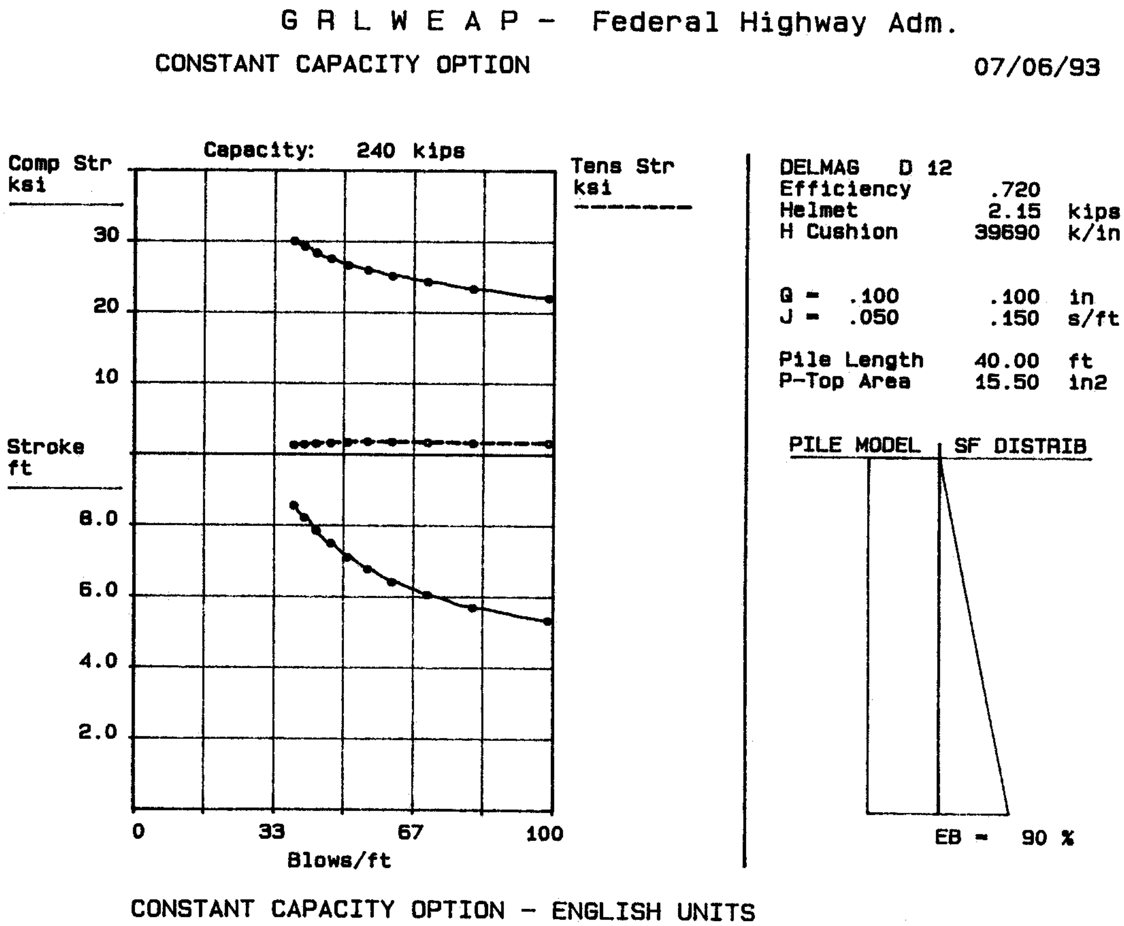


Figure 9-6: Graph of stroke versus blow count for a constant pile capacity

The stresses predicted by the wave analysis should be compared to safe stress levels. This comparison is usually performed for the tensile and compressive stress shown at the computed driving resistance for the estimated pile length. FHWA publication RD 83-059, "Allowable Stresses in Piles", presents detailed information on dynamic loading of piles. From that report FHWA has developed the following limiting stress levels to prevent pile damage during driving.

Pile Type	Allowable Driving Stress
Steel	0.9 Fy
Concrete	(0.85 F′c – effective prestress) in compression
	(3 √F′c + effective prestress) in tension
Timber	3 F′a (not to exceed 3000 psi)

Where: Fy = Yield strength of steel
F′c = 28 day concrete cylinder strength
F′a = allowable compressive stress of timber including allowance for treatment effects

The last operation in pile design is to insure that the pile can be driven to the estimated length without damage. For this purpose a trial wave equation analysis is done with an appropriately sized hammer. Figure 9-7 can be used to choose a reasonable hammer for wave analysis.

In general Figure 9-7 hammer energies are lower than the optimum energy necessary to drive the appropriate pile cross section. Judgement should be used in selecting the hammer size. If initial wave equations yield high blow counts and low stresses the hammer size should be increased. ~~In design wave equation analysis the designer should determine if a reasonable range of hammer energies can drive the proposed pile section without exceeding both the allowable driving stresses (above) and a reasonable range of hammer blows, ie 30 to 144 for friction piles and higher blows of short duration for end bearing piles.~~

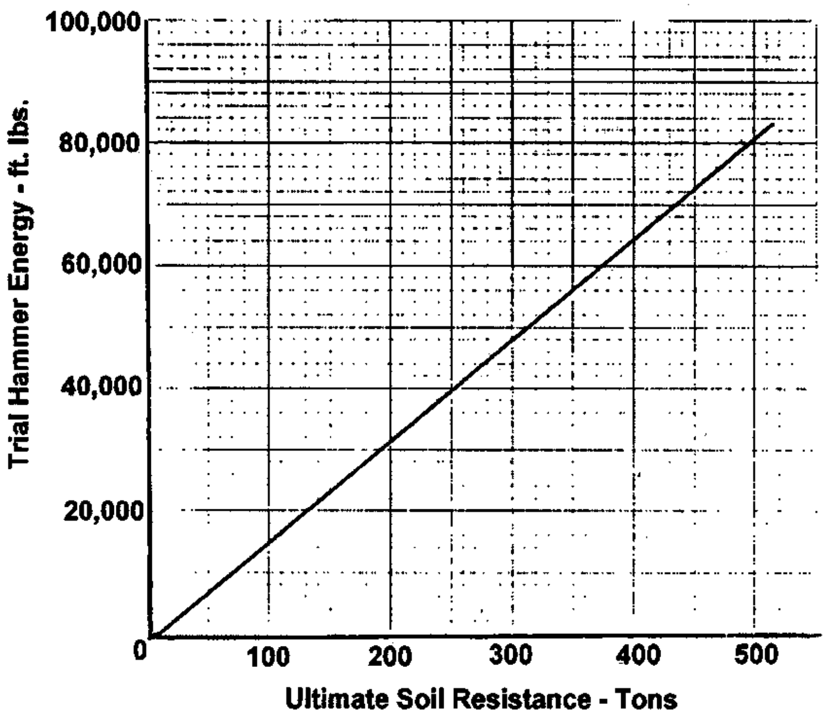


Figure 9-7: Suggested trial hammer energy for wave equation analysis

Example 9-1: Determine If The 14″ Square Concrete Pile Can Be Driven To A Driving Capacity Of 225 kips By Using The Wave Equation Output Summary. Assume The Concrete Compressive Strength Is 4000 psi And The Pile Prestress Force Is 700 psi.

WAVE EQUATION OUTPUT SUMMARY

R_{ult} kips	Blow Count BPF	Stroke (EQ) Ft.	Tensile Stress Ksi	Compressive Stress Ksi	Transfer Energy Ft-Kip
35.0	7	3.27	-0.73	1.68	13.6
80.0	16	3.27	-0.32	1.71	13.6
140.0	30	3.27	-0.20	1.73	13.0
160.0	35	3.27	-0.14	1.73	13.0
195.0	49	3.27	-0.00	1.75	12.8
225.0	63	3.27	0.0	1.96	12.7
280.0	119	3.27	0.0	2.34	12.6
350.0	841	3.27	0.0	2.75	12.5

Solution:

Acceptable driveability depends on achieving the hammer blows between 30 and 144 at the driving capacity, and assuming that the allowable compressive and tensile driving stress are not exceeded.

1. At R_{ult} = 225 kips, blow count = 63 O.K. between 30 and 144

2. For concrete piles, the allowable driving stresses are:
   - Compressive stress allowed = 0.85 F′c – prestress = 3400 – 700 = 2700 psi, actual maximum compressive stress up to 225 kips from wave equation output summary is 1.96 ksi or 1960 psi ≤ 2700 psi allowed value. O.K.
   - Tensile stress allowed = $3\sqrt{F'c}$ + prestress = 190 + 760 = 890 psi, actual maximum tensile stress up to 225 kips from wave equation output summary is 0.730 ksi or 730 psi ≤ 890 psi allowed value. O.K.

Therefore driveability is acceptable.

9.8 PILE CONSTRUCTION CONTROL CONSIDERATIONS

The approval of a contractor's driving equipment is an example of design and field coordination. The recommended procedure uses the results of a wave equation analysis to determine if the contractor's equipment is adequate to drive the pile to the estimated length without pile damage. The steps in this procedure are as follows:

1. The pile specifications should include a statement similar to:

   "All pile driving equipment to be furnished by the contractor shall be subject to the approval of the engineer. Prerequisite to such approval, the contractor shall submit the following:

   a. A completed pile and driving equipment data form (Figure 9-8) for each hammer proposed for the project.

   b. A wave equation analysis performed by a professional engineer for each proposed hammer to the soil resistance value listed on the plans.

   Contractor notification of acceptance or rejection of the hammer will be made within 14 days of receipt of the data form and wave equation analysis."

2. The designer should also receive a copy of the data form and the wave equation results. An independent wave equation should be done to verify the submitted results and to establish driving criteria for the piles. In approving equipment, the designer enters the pile driving information form with the soil resistance to be overcome during driving and establishes if a reasonable number of blows per foot are required to attain that resistance. For friction piles, 30-144 blows per foot are considered reasonable. Higher blow counts can be permitted for end bearing piles since the duration of high blow counts is short. Then the stresses at that blow count are checked to determine if the values are below the allowable driving stress of the pile material. If these items are satisfied, the equipment can be approved and the information sent to the field engineer. The wave equation results may be transmitted to the field with a recommendation to reject or approve the hammer. If

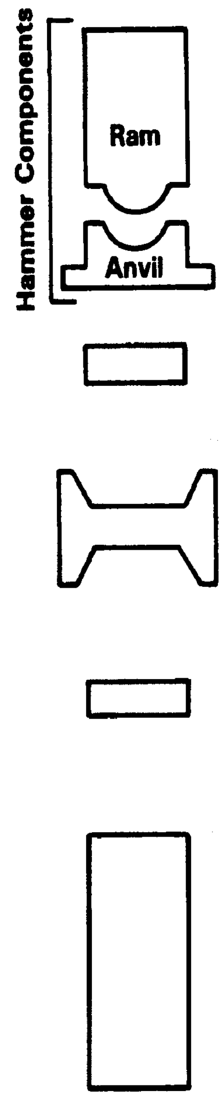
Contract No.: ______________________ Structure Name and/or No.: ______________________

Project: ______________________ ______________________

______________________ Pile Driving Contractor or Subcontractor: ______________________

County: ______________________ ______________________

(Piles driven by)



Hammer

Manufacturer: ______________________ Model: ______________________

Type: ______________________ Serial No.: ______________________

Rated Energy: ____________ at ____________ Length of Stroke

Modifications: ______________________

____

____

____

____

Hammer Cushion

Material: ______________________

Thickness ______________________ Area: ______________________

Modulus of Elasticity – E ______________________ (P.S.I.)

Coefficient of Restitution-e ______________________

Drive Head

Helmet / Bonnet / Anvil Block / Pile Cap – Weight: ______________________

Pile Cushion

Cushion Material: ______________________

Thickness: ______________________ Area: ______________________

Modulus of Elasticity – E ______________________ (P.S.I.)

Coefficient of Restitution ______________________

Pile

Pile Type: ______________________

Length (In Leads) – ______________________

Weight/ft. ______________________

Wall Thickness: ______________________ Taper: ______________________

Cross Sectional Area ______________________ in^2

Design Pile Capacity: ______________________ (Tons)

Description of Splice: ______________________

____

Tip Treatment Description: ______________________

____

Distribution
One Copy Each To:
- ☐ State Bridge Engineer
- ☐ State Soils Engineer
- ☐ District Engineer
- ☐ Resident Engineer

Note: If mandrel is used to drive the pile, attach separate manufacturer's detail sheet(s) including weight and dimensions.

Submitted By: ______________________ Date: ______________________

Figure 9-8: Pile and Driving Equipment Data Form the hammer is recommended for approval, the transmittal will contain pile driving criteria.

3. The procedure for hammers proposed for substitution during the contract is the same.

During production operations, the engineer will check to see if the necessary blow count is attained at the estimated length shown on the pile driving information form. The resistance is generally acceptable if the blow count is within 10 percent of that expected, or if the expected blow count is achieved within 5 feet of the estimated length. The field engineer should be aware that blow counts higher than expected will cause an increase in pile stress the correlation of blow count and stress is shown on the information form. If necessary an upper blow count limit may need to be established to prevent damage.

Most important, if either radically different blow counts (higher or lower) or damage are observed at the estimated length, the foundation engineer should be contacted immediately. The phone number of the foundation engineer should be on the information form.

All engineers should realize that pile driving is not by any means an exact science and actual blow counts may be expected to vary somewhat even in the same footing. The objective of construction control of pile driving is to insure that the pile is capable of safely carrying the design load. This means that the driven pile is not damaged and enough soil resistance is mobilized for support. Both these items can be checked from the pile wave analysis output.

The pile wave equation analysis is a big improvement over dynamic formulas because the variables of pile length and flexibility are accounted for in addition to the variations in the contractor's driving system and the project soils. A wave equation program entitled "WEAP" was developed for FHWA in 1975 and updated periodically thru 1987 when FHWA development activities ceased. However, since 1988 numerous technical changes and programming improvements have been made periodically to the WEAP model by GRL and Associates Inc. The resulting proprietary program, GRLWEAP, has been accepted by public agencies for use on a variety of public projects since 1988.

The use of wave equation in construction control provides the engineer with a prediction of the behavior of the driven piles during installation. While this prediction is superior to previous methods of estimating driveability, the optimal method of determining pile driveability is to obtain dynamic measurments during pile installation. Dynamic test methods commonly employ accelerometers and strain gages, attached to the pile during driving, to measure real time strains and accelerations produced during the driving process. Field computers use these measurements to output information, which the inspector can use to:

- Monitor hammer and driving system performance,
- Evaluate driving stresses and pile integrity, and,
- Verify pile capacity

Additional details of the dynamic test procedure are shown in section 9.13.

9.9 DEEP FOUNDATION SPECIFICATIONS

9.9.1 Pile Specification

Early pile specifications placed the major responsibilities for pile capacity determination on the field staff. Little analysis was done in design to provide accurate estimates of the required pile length to safety support the design load. No design analyses were done to account either for the actual soil resistance to be overcome to drive the pile to the estimated length or the stresses generated in the pile during driving. Specifications frequently placed the responsibility of determining what pile length to order on the contractor. Delays for reordering additional lengths or splices to reach final tip were considered incidental to the price bid for the item. The result was higher bid prices to account for the risks involved with the pile item.

At present, procedures, equipment and analysis methods exist to permit the designer to accurately establish pile length and section for any driving condition. Basic foundation design procedures are routinely followed by nearly all public agencies. Yet much of the available design information is neither reflected in the pile specification of the agency nor utilized by the construction staff. Many agencies perform detailed static analyses to determine pile length, but control the pile length actually installed in the field with the Engineering News Formula which is known to be the least accurate and least reliable of all dynamic formulas. Improvements are required in the pile specifications to permit the cost effective use of the state-of-the-art pile techniques. A model driven pile specification is included in the FHWA Geotechnical Engineering Notebook which incorporates the improvements listed below.

1. Ordered Length Replaces Estimated Length - Public highway agencies should assume responsibility for determining and placing in the contract documents the pile length necessary to safely support the design load. Extra costs associated with overruns or underruns due to inaccurate length determination should not be borne by the contractor. The concept of a fair pile specification is based on the highway agency performing adequate subsurface work and design analyses to rationally establish pile lengths during the design phase.

2. Driving Capacity Replaces Design Load - Installation of piling to a predetermined length involves overcoming the design soil resistance multiplied by the safety factor in the bearing layer plus the resistance in any overlying layers unsuited for bearing. The use of procedures involving only design load, such as the Engineering News Formula, should be replaced with ultimate load based methods. The ultimate load to be achieved should be based on both the actual resistance to be overcome to reach ordered tip and the confidence in the method of construction control to be used. Ultimate values are now required for load-resistance factor design procedures adopted by AASHTO.

3. Increased Emphasis on Approval of Driving Equipment - The use of properly sized pile driving equipment will practically insure a successful installation of properly designed piles. Conversely, improperly sized pile driving equipment insures a pile project fraught with problems regardless of how well the pile design was done; to small a hammer results in extremely difficult, time consuming driving; too large a hammer results in pile damage. Fair pile specifications should place great emphasis on a formal approval procedure for the hammer and hammer appurtenances. This approval procedure is the most significant improvement which could be made to current specifications.

4. Field Control of Pile Capacity by Wave Equation and Dynamic Pile Testing to Replace Engineering News Formula - Current good piling practice includes the use of the wave equation and dynamic

pile testing in place of dynamic formula to monitor pile driving for all projects. In particular, continued use of the Engineering News Formula can only result in unreliable, costly pile foundations. Highway agencies need to utilize modern methods both in design and construction control of pile foundations. The wave equation is designed to use driving resistance, basic soil properties and calculated pile lengths in conjunction with driving equipment characteristics to produce both the necessary hammer blow count for the desired load and the maximum pile stress to be encountered during driving. Dynamic pile testing provides a quick, reliable field test alternate to static load testing as well as a supplement to pile wave equation analysis.

5. Separation of Payment into Fixed and Variable Cost Items to Replace Lump Sum Items - Fair compensation for work performed in pile driving can only be accomplished by recognizing and providing bid items for those contract costs which are fixed and those contract costs which are variable. Some payment methods used by highway agencies involve lumping fixed and variable costs into a single item. Such lump sum items with variable contingencies are recognized as high risks items by contractors whom, to avoid a monetary loss, increase the price bid to cover the risk. An example of this situation is lumping the cost of pile points into the per foot cost of the pile. Pile points are a fixed cost item. However, when lumped with pile length, the pile point becomes a variable cost item, i.e., the contractor breaks down the point cost to a per foot cost based on estimated length. If an overrun in length occurs the price paid to the contractor for the point increases; conversely an underrun results in an underpayment for the pile point cost.

9.9.2 Drilled Shaft Specification

Construction control of drilled shaft work requires that the criteria for field measurements and tests be clearly outlined in the construction specification. However, the most important item to check for a successful project is the qualifications of the drilled shaft contractor. Drilled shaft construction is a specialty item. Specifications must include a procedure for establishing that the contractor possesses both proper tools and expertise to install the size of drilled shaft designed for the project. Projects involving difficult drilling conditions, large diameter (>8') shafts, non-redundant shaft designs, or over-water shaft installation require special expertise. Specifications should require either submittals of qualifications at the time of bid or an on-site demonstration of contractor abilities by constructing to specification a trial drilled shaft previous to installing production shafts.

The specification should communicate specific construction control items which directly relate to the shaft design, i.e., construction requirements for end bearing shafts differ from friction shafts. A discussion follows of general items to be included in the specification. A model drilled shaft specification is included in the FHWA Geotechnical Engineering Notebook.

1. Construction Method - The construction methods to be permitted on a specific project are directly related to the method of load transfer assumed in the project design. The type of drilling method, presence of permanent casing, and clean out procedure all affect the drilled shaft load transfer behavior in skin friction and end bearing. For instance, the permanent casing method cannot be permitted in subsurface deposits which were designed for full mobilization of shaft skin friction with the soil.

   Fortunately, numerous combinations of equipment and procedures are commonly available to permit successful installation of drilled shafts for any stated design criteria. Specifications should not needlessly restrain contractors in their choice of tools, equipment or construction methods. The key to cost effective projects is permitting flexibility in contractor operations to achieve the design intent; particularly at sites where variable subsurface conditions are expected.

Quality of the end product is monitored and controlled by including explicit definitions and controls in the following areas: installation plan, tolerances, acceptance and rejection criteria, and project documentation. The success of specification relies heavily on responsible and knowledgeable inspection, and experienced drilled shaft contractors. Even the most conservative design can result in problems if: the specified construction procedure is inappropriate for the project conditions, the inspection is not effective, or the contractor is poorly equipped or inexperienced with drilled shaft construction.

2. Drilling Slurry - Drilling slurry is an effective method of stabilizing drilled shaft excavations until either a casing has been installed or concrete has been placed. The properties of drilling slurry should be both monitored and controlled prior to and during the drilling, and prior to concrete placement. Primary concerns connected to slurry use are: the shape of the borehole be maintained during the excavation and concrete placement; the slurry does not weaken the bond between the concrete and both the natural soil and rebar; all of the slurry is displaced from the borehole by the rising column of fresh concrete; and any sediment carried by the slurry is not deposited in the borehole.

   The engineer's concerns regarding the behavior and effectiveness of slurry projects can be satisfied by appropriate specification requirements. These requirements include: specifying a suitable range of slurry properties both prior to and during excavation and prior to concreting; performing slurry inspection tests; and construction of preproduction trial shafts by the slurry method.

3. Payment for Shaft Excavation - The ability of a contractor to excavate a particular strata depends on the type, size, and condition of the contractor's equipment, as well as, the skill of the equipment operators. Two alternate methods of payment for shaft excavation can be specified; unclassified payment and classified payment. Both methods require separate compensation for obstruction removal. A single unclassified excavation item alternate was included primarily to avoid ambiguous, unfair and impractical definitions of soil and rock excavation which have been used by some agencies. The engineer's concern regarding high contingency bids, when using this method, can be satisfied by performing an adequate site investigation and making this information available to bidders.

   A. Unclassified Payment

      Unclassified payment is appropriate at sites where a comprehensive exploration program has been completed specifically for the drilled shaft foundation. Such a program should include a full size inspection shaft in representative subsurface areas and a test boring to beyond the anticipated shaft depth at the following intervals:

      i. Non-Redundant (Single) Shaft Foundations

         One boring per shaft;

      ii. Redundant (Multiple) Shaft Foundations

Shaft Diameter	Guideline Boring Requirement
72" or greater	1 boring per shaft
48" - 72"	1 boring per 2 shafts
less than 48"	1 boring per 4 shafts

The boring logs and inspection shaft logs should contain specific information about equipment used and rate of penetration in addition to soil, rock, obstructions and water conditions. All the aforementioned information should be made available to bidders as part of the contract documents.

B. Classified Payment

Separate items for classified payment should be used for all other projects and defined in terms of standard excavation and special excavation. Standard excavation includes hole advancement with conventional augers fitted with either rock or soil teeth, drilling buckets, and/or underreaming tools. Special excavation is paid when the hole cannot be advanced with conventional tools. Hole advancement under special excavation requires special rock augers, core barrels, air tools, blasting or other methods of hand excavation. All earth seams, rock fragments or voids which are encountered after special drilling commences are paid as special excavation. Obstructions which require unconventional excavation techniques are not considered special excavation for payment but paid under a separate item.

4. Special Bidding Requirement - Drilled shaft costs are controlled by the character of the subsurface materials encountered during excavation. No drilled shaft contractor should be permitted to either bid drilled shaft work or act as a subcontractor to a bidder unless he has: visited the site, inspected soil and rock samples (if made available in the contract documents by the agency) and received the subsurface information made available in the contract documents.

9.10 DEEP FOUNDATION LOAD TESTS

A static load test is conducted to measure the response of a deep foundation under applied load. Conventional static load test types include axial compressive, axial tensile and lateral load testing. The cost and engineering time associated with a load testing program should be justified by a thorough engineering analysis and foundation investigation. Load tests are possible on either single elements or groups but due to cost considerations only single element tests are generally performed on production projects. The FHWA has published the results of pile group load tests in sands and clays in FHWA TS-87-221 and 222.

Static load tests provide the best means of determining deep foundation capacity and if properly designed, implemented and evaluated, should pay for themselves on most projects. Depending on availability of time and on cost considerations, the load testing program may be included either in the design or in the construction phase. Dynamic load tests, performed in conjunction with static load tests, greatly increase the cost-effectiveness of a pile load test program and should be specified whenever piles installed by impact driving are load tested.

Many different procedures have been proposed for conducting load tests. The main differences are in the selection of loading systems, instrumentation requirements, magnitude and duration of load increments, and interpretation of results. Some innovative test procedures which are potentially applicable to piles and drilled shafts include the Osterberg load cell and high strain dynamic testing such as the Statnamic test. The Osterberg procedure involves installation of a non-retrievable hydraulic jack at the pile or shaft base. The jack reacts against the larger of the base or skin resistance to cause a failure condition of the weaker resistance. High strain tests involve the use of heavy drop weights or explosive devices (the Statnamic procedure) to create strain and acceleration data which are used to predict capacity.

The purpose of load testing is:

- To develop criteria to be used for the design and installation of the pile foundation, or

- To prove the adequacy of the pile-soil system for the proposed pile design load.

9.10.1 Prerequisites for Load Testing

Load testing is not a substitute for an adequate foundation investigation program. In the planning stage of any load test program, the following will be required:

- Adequate subsurface exploration.

- Well-defined subsurface profile.

- Adequate soil/rock testing to determine engineering properties.

- Static analysis results to rationally select foundation type and length, as well as the load test site(s).

9.10.2 Advantages of Load Testing

Load testing offers several advantages:

- Allows a more "rational" design. The load transfer can be determined much more reliably by applying a test load to a foundation element than from the results of laboratory tests or based on assumptions.

- Allows use of lower factor of safety. Many foundations are designed using a factor of safety of 3. Testing allows the engineer to use a lower factor of safety which translates into cost savings.

- Improved knowledge regarding load transfer has the potential of permitting an increase in the design load and a reduction in the foundation number or length (for friction elements) with a corresponding savings in foundation costs.

- Verifies that the design load can be attained at selected tip elevation.

The reasons often cited for not load testing include:

- Costs involved.

- Delays to contractor if done as part of construction contract.

- Delay of project if done in the design phase.

The cost of performing a load test should always be weighed against the benefits to be obtained. A load test costing $100,000 could be considered inexpensive if cost savings in the millions resulted. Delay of a project during the design or construction phase is most likely to occur in those instances where the decision to perform load tests is made at the last minute. The need for design phase load tests should be addressed in the early stages of the design phase, and construction phase load tests should be clearly specified in the contract documents. In this way, the load tests are incorporated into the schedules and unforeseen delays are minimized.

9.10.3 When to Load Test

The decision whether or not to initiate a load test program on a particular project will be influenced by several factors. The following criteria can be used to assess when load testing can be effectively utilized:

- When the potential for substantial cost savings is readily apparent. This is often the case on large projects, either to determine whether friction pile lengths can be reduced, or whether allowable pile stresses can be increased for end-bearing foundations.

- When safe load carrying capacity is in doubt, due to limitations of an engineer's experience or unusual site or project conditions.

- When soil or rock conditions vary considerably from one portion of a project or another.

- When the design load is significantly higher than typical design loads.

- When time related pile-soil capacity changes are anticipated (i.e., setup or relaxation).

- When using precast concrete friction piles - so that piles can be cast long enough to avoid costly and time consuming splicing during construction.

- When new, unproven pile types and/or pile installation methods are utilized.

- When existing foundations will be utilized to support a new structure carrying heavier loads.

- When a reliable assessment of uplift resistance for lateral behavior is important.

- When, during construction, the load carrying capacity of a pile by hammer formula or dynamic analysis differs from the estimated ultimate load at the anticipated tip elevation (for example, H-piles that "run" when driven into loose to medium dense sands and gravels).

9.10.4 Effective Use of Load Tests

1. During Design

   On major projects, the benefit to construction of conducting a load test program in the design phase should be considered. The subsurface profile must be adequately defined to determine the optimal number and locations of load tests as well as the area over which each test can be considered representative for driving of production elements. A design phase static load test program will require highway agencies to prepare and let a construction contract. The unit cost per test will be significantly higher than for tests performed during construction (particularly if over-water testing is involved), due to the mobilization of men, materials and equipment to install a small number of piles. For maximum benefit, the design load test program should be completed at least a year before project advertisement to permit foundation and structural engineers to optimize final design.

   Design phase load tests offer several advantages:

   - Allow load testing of alternate foundation types and selection of most economical foundation.

   - Installation information can be made available to bidders - this should reduce their bid "contingency,"

   - Greatly reduce potential for claims arising from pile driving or shaft installation problems, especially for piles which are difficult to splice.

   - Maximize cost savings for foundations (e.g., permit lower factor of safety, permit changes in design load and number of elements, reduce number of orders-on-contract).

2. During Construction

   Typically, the primary purpose of load tests performed during construction is to verify that the design load does not exceed allowable capacity (proof testing), particularly if set-up or relaxation is anticipated. For drilled shaft and piles installed other than by driving with an impact hammer (e.g., vibrated or auger cast), load tests during construction can be used to confirm that both the soil and the structural foundation element can safety sustain the design load.

   Construction phase load tests are also commonly used to determine final tip elevation of production piles after test drive (indicator) piles are evaluated at estimated length.

3. Limitation of Load Tests

   A load test performed on a single pile does not:

   - Account for long-term settlement

   - Take into account downdrag from settling soils

   - Take into account the effect of group action

   - Eliminate the need for an adequate foundation investigation.

The above must be considered when using load test results to design or analyze deep foundations.

9.11 QUICK LOAD TEST METHOD FOR STATIC TESTING

The "Quick Load Test Method" is the recommended method for load testing of piles and drilled shafts on highway projects. The quick load test, originally developed by the Texas Highway Department, is allowed as an optional load test procedure by ASTM D-1143. Basically, this method requires that load be applied in increments of 10 to 15 percent of the design load with load, gross settlement, and other pertinent data recorded immediately before and after the addition of each increment of load. After an increment of load is added the load is maintained constant for a time interval of 2-1/2 minutes before the next increment is added.

The Quick Load Test Method offers the following advantages:

- The load test can be performed in 1-2 hours, versus over 100 hours in the standard AASHTO method, with resultant savings in time and money.

- Construction delay to the project caused by load testing is greatly reduced.

- Full-scale load testing on smaller projects is feasible because of reduced time and costs.

- Simplicity of the testing procedure ensures standardization of the test and easy interpretation and utilization of the results.

Similar advantages can be achieved by using the constant rate of penetration (CRP) load test procedure which is described in ASTM D-1143.

9.11.1 Factor of Safety - Static Load Test

To obtain the allowable load, the ultimate failure load determined from a load test should be divided by a factor of safety of at least 2.0. Larger factors of safety may be required:

- -For friction piles in clay, where group settlement may control the allowable load.
- Where total settlement that can be tolerated by structure is exceeded.

9.11.2 Rule of Thumb for Piles - Cost-Effectiveness of Quick Load Test

It is difficult to decide how large a project has to be before a pile load test is likely to be cost-effective. On projects where friction piles will be used, experience has shown that load tests will typically show that pile lengths can be reduced at least 15 percent versus lengths that would be required by ENR formula. Therefore, this 15 percent pile length reduction, can be used to establish a simple rule of thumb formula to compute the estimated pile footage which the project must have to make the load test cost effective. This formulas is (0.15) (Cost/L.F. of Pile) (X) = Cost of Load Test.

Where X = The minimum estimated pile footage, the project must have before the load test would probably at least pay for itself.

Example 9-2:

Assume a project will require 100 ton design load piles. Estimated pile cost is \$15/L.F. (furnished and driven). Estimated cost to perform a Quick Load Test is \$15,000. How much estimated pile footage must the project contain for a load test to be cost effective?

$$X = \frac{\$15{,}000}{(0.15)(\$15/\text{L.F.})} = 6{,}700\,\text{L.F. of Pile}$$

Thus, the project would have to contain an estimated 7,000+ lineal feet of piling for load testing to provide a potential cost savings.

9.11.3 Load Testing Details

For information and specifications for compressive, tensile, and lateral load testing consult FHWA Publication SA-91-042, "Static Testing of Deep Foundations."

9.12 DAVISSON'S LIMIT

The Davisson limit was developed in conjunction with the wave equation analysis of driven piles and is gaining widespread use. It is primarily intended for use with load test results from driven piles tested in accordance with quick methods.

Davisson's limit is defined as the load corresponding to the movement which exceeds the elastic compression of the pile by a value of 0.15 inches plus a factor equal to the diameter of the pile divided by 120. (For example, for the 18 inch diameter pile shown in Figure 9-9: x = 0.15 in. + (18 in. / 120) = 0.3 in.) Piles exceeding 24 inches in diameter require modification of the limit value to the elastic compression plus the diameter divided by 30.

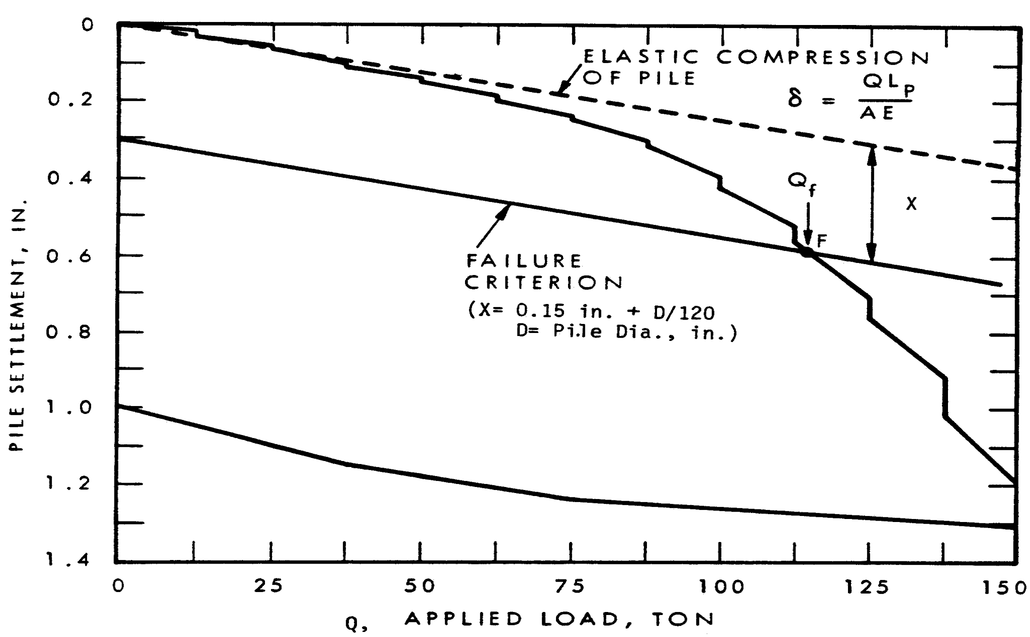


Figure 9-9: Alternate Method Load Test Interpretation (After Davisson)

9.13 DYNAMIC PILE LOAD TESTING

Dynamic pile load testing is the estimation of static axial compressive pile capacity from dynamic measurements of pile strain and acceleration. Before the start of testing, two strain transducers and two accelerometers are securely attached to opposite sides of the pile near its top. These gages are connected to the pile analyzer (Figure 9-10). A separate device, such as an oscilloscope may be used to display the data being analyzed and a portable magnetic tape recorder to store the data. These functions are integrated into post 1991 versions of the pile analyzer.

As the pile is struck by a pile hammer, the strains and accelerations detected by the corresponding gages on the pile are converted by the pile analyzer into forces and velocities. The latter quantities are processed to obtain an estimate of the static pile capacity at the time of testing and for pile design. The additional information obtained and displayed includes compressive and tensile stresses in the pile, transferred energy to the pile, and the force and velocity at the top of the pile throughout the duration of the hammer impact. An experienced operator can use this data to evaluate the performance of the pile driving system and the condition of the pile. Usually the results of the dynamic testing are enhanced by performing a computer analysis known as the Case Pile Wave Program (CAPWAP) to verify the correctness of assumed dynamic inputs such as damping.

ASTM D4945-89, Standard Test Method for High-Strain Dynamic Testing of Piles, contains a detailed description of the equipment requirements and test procedure for dynamic pile load testing.

9.13.1 Applications

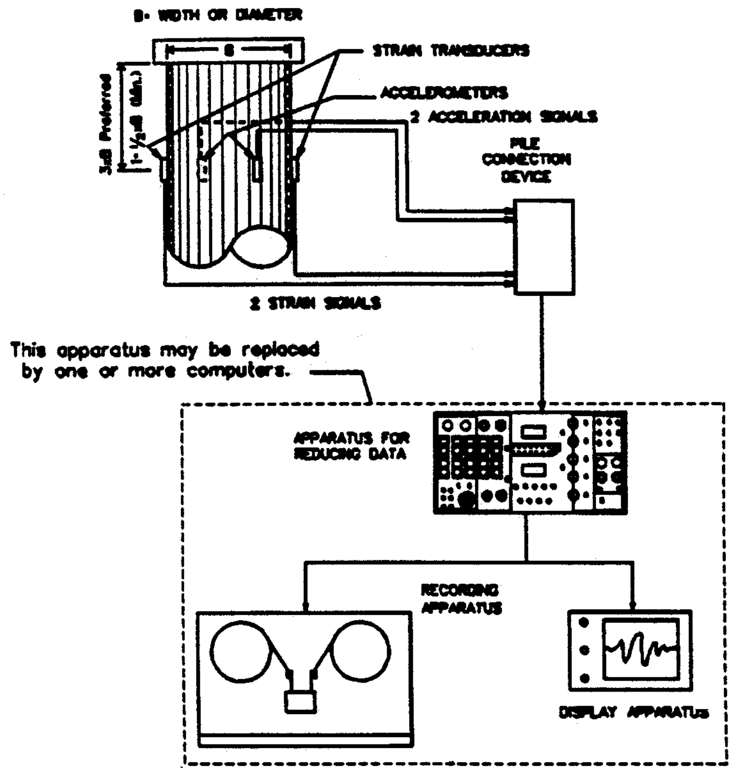
Dynamic pile load testing costs much less and requires less time than static pile load testing. Important information can be obtained regarding the behavior of both the pile-soil system and the pile driving system that is not available from a static pile load test. Consequently, dynamic pile load testing has many applications:

- As a supplement to static pile load testing on major projects, thereby permitting a reduction in the number of static tests.

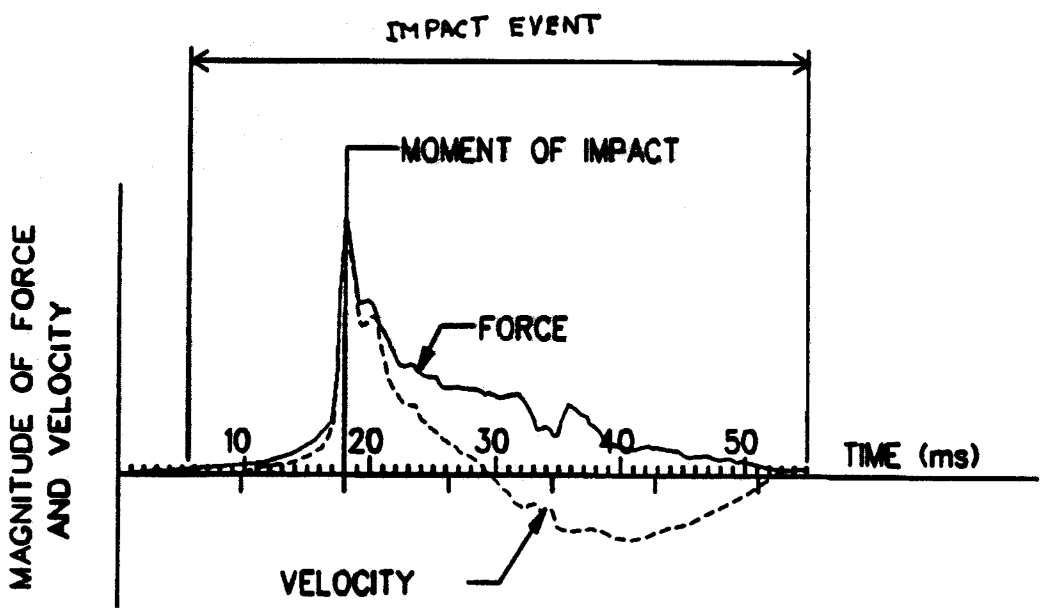
- On small-scale projects where static pile load tests are difficult to justify economically.

- On projects, such as over-water installations, where full-scale static pile load tests are not feasible.

- To monitor driving stresses and pile integrity.



SCHEMATIC DIAGRAM FOR APPARATUS
DYNAMIC MONITORING OF PILES



Typical Force and Velocity Traces Generated by the Apparatus for Obtaining Dynamic Measurements

Figure 9-10: Schematic Diagram for Apparatus Dynamic Monitoring of Piles

- To evaluate hammer performance.

- To validate wave equation input values.

Dynamic measurements can also be obtained from high strain testing which may be applicable to non-driven piles and drilled shafts. However, consideration should be given to structural damage of the foundation element because the test requires a hammer blow that has sufficient energy to mobilize the capacity of the soil surrounding the foundation element.

9.13.2 Interpretation of Results and Correlation with Static Pile Load Tests

The results of dynamic pile load test should be interpreted by an experienced tester who has had the opportunity to observe and evaluate the results from many dynamic load tests and can detect the signs, not always readily apparent, of unusual soil-pile response, pile damage, erratic hammer operation or testing equipment malfunction.

Interpretation of the results of dynamic pile load tests also requires an awareness of the differences in behavior of dynamically and statically loaded piles. Improper correlations of dynamic and static pile load test may be caused by the following:

- Incorrectly assumed soil damping parameters. This source of discrepancy can be minimized by performing a computerized analysis to match measured and computed relationships between force and velocity to determine the most appropriate damping parameter.

- Time-related changes in pile capacity. Depending on soil type and pile characteristics, the capacity of a pile may increase or, less commonly, decrease with time. The principal causes are time-related changes of pore water pressure in the soil. The effects can be assessed by restriking the pile at various time intervals after driving and comparing the capacity against the driving capacity obtained during the initial drive. The pile capacity should be determined during the first few blows of the re-strike. When comparing the results of dynamic testing against those of a static pile load test, at least one dynamic test should be performed after completion of static testing.

- Pile tip displacement during dynamic testing may be inadequate to mobilize full end bearing. Frictional resistance between a pile and the surrounding soil is mobilized at a fraction of the pile movement necessary to mobilize full end bearing resistance. A penetration resistance of 10 blows/inch or higher, may produce insufficient strain in the soil to mobilize full end resistance. This results in an underestimate of the end bearing capacity. For many types of piles, the estimate can be improved by performing a force-velocity match both for the initial drive and for the restrike data. The tip capacity derived from the initial drive is combined with skin resistance from the restrike to obtain the total pile capacity. However, this method may not be applicable for open-ended pipe, H-piles, and precast cylinder piles. In the case of these piles, only the structural area of the pile can mobilize the toe bearing during installation. This is a significantly lower value than what may be experienced in the static load test, since the soil will adhere to the pile with time and create a plug.

  If dynamic pile load tests are performed and interpreted by experienced and knowledgeable testers, the correlation between pile capacities determined from static and dynamic pile load tests is good, i.e., $\pm$ 15 percent. The correlation would not be as good for open-ended and H-piles. However, dynamic load tests on these types of piles would, in general, underestimate the static pile capacity.

9.14 ADDITIONAL LOAD TEST METHODS

Two methods of load testing have been introduced in recent years which have been used to varying degrees by highway agencies; the Osterberg Cell and the Statnamic methods. Although the details of each method are beyond the scope of this manual, a short primer follows on each method. For additional details the reader should consult other FHWA publications such as FHWA-HI-97-014, "Design and Construction of Driven Pile Foundations".

9.14.1 The Osterberg Cell Method

A recent development for evaluation of deep foundation capacity is the Osterberg Cell test. This test employs a sacrificial pressure jack that is either placed in a open-end foundation element (drilled shaft or pile) or attached to the base of a pile prior to driving. This proprietary device has proven to be a simple and efficient method of applying static load to a deep foundation. As shown below the Osterberg cell uses the ground as a reaction for the test load rather than the common static test that relies on an external reaction system.

The cell has been used at both the base and at intermediate levels in open-end foundation elements. In the case of drilled shafts the cell is commonly attached to the rebar cage and lowered into the hole. In the case of piles, the cell is attached to driven displacement type piles such as closed-end pipe or concrete piles prior to installation. The cell may be installed after driving either open-end pipe piles or mandrel driven piles.

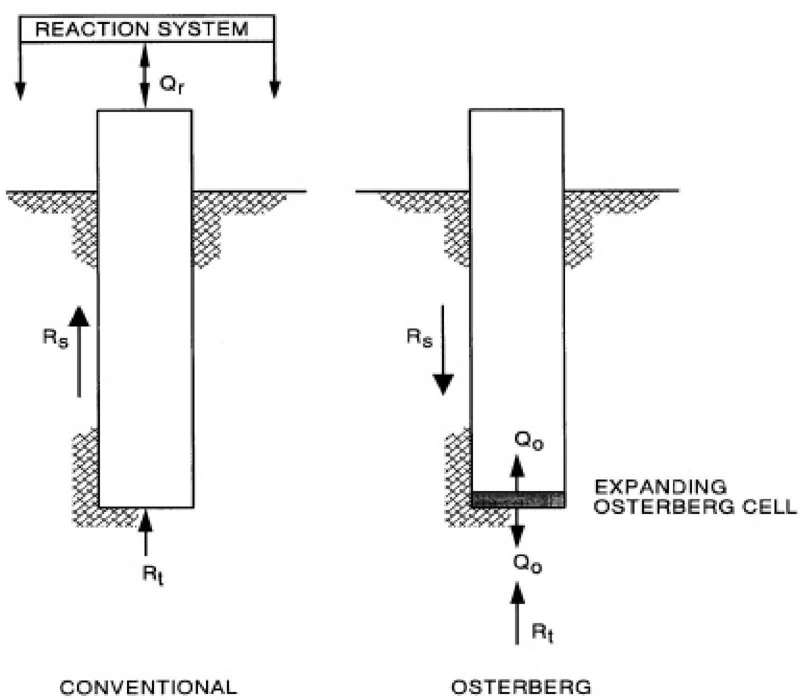


Figure 9-11: Comparison of reaction mechanism between Osterberg Cell and Static Test

As of late 1999, the Osterberg cell is manufactured in a variety of sizes for both open-end installations and driven pile installations. The tables below, which were provided by the American Equipment and Fabricating Corporation, East Providence, RI 02914, contain pertinent dimensions and load capacities for Osterberg cells. Table A refers to cells placed in previously installed foundation elements; Table B refers to driven piles.

Table A – Osterberg Cells (In-place) 1999

Size	Diameter Inches	Height Inches	Capacity Tons	Weight Pounds
5	5.25	5.18	75	32
9	9.00	10.75	200	190
13	13.00	11.65	400	300
21	21.25	11.65	1200	800
26	26.25	11.65	1800	1230
34	34.25	12.37	3000	2015

Table B – Osterberg Pile Load Cells 1999

Capacity - Tons	Size - Inches	Stroke - Inches	Description
200	14	6	Round-Steel pipe
300	14	6	Square-Precast Concrete
900	18	8	Round-Steel pipe
950	30	9	Square-Precast Concrete

The Osterberg Cell test does have some limitations in that the total failure load of the foundation element is not usually measured; only the failure load of the friction above the cell or the resistance below the cell are measured. Failure can be inferred by extrapolating the non-failure portion of the test in some cases. The Osterberg Cell test has not been standardized by AASHTO or ASTM as of 1999. Additional information on the Osterberg Cell test can be found in FHWA publication FHWA SA-94-035 or at www.loadtest.com

The Osterberg Cell has been used in a variety of soil and rock conditions. The cell has been used to determine the bond stress in rock sockets or in dense glacial tills. In these applications, the use of a caliper device is highly desirable to determine the exact dimensions of the drilled socket. The actual dimensions of holes drilled into intermediate geomaterials can vary from the diameter of the drilling tool due to a variety of geologic factors or drilling considerations. Calipers are available in either mechanical or electronic configurations. In addition, a variety of strain gage devices have been used in conjunction with the Osterberg Cell test to develop a distribution of resistance along the foundation element. Such measurements can also be taken below mid-height cells by extending instrumented rebar below the base of the cell.

9.14.2 The Statnamic Test Method

Another recent development for evaluation of foundation load capacity is the Statnamic test method. The Statnamic method is a proprietary method developed by the Berminghammer Foundation Corporation (www.berminghammer.com). A new ASTM draft standard, entitled "Standard Test Method for Piles Under Rapid Axial Compressive Load", has been proposed but had not been approved as of 1999.

The Statnamic test method uses solid fuel burned within a pressure chamber to create a rapidly increasing pressure between a reaction mass and the top of the foundation element. As the gas pressure increases, the reaction mass is accelerated upward and the foundation element is forced into the ground. After the maximum pressure is achieved, the pressure vents and the element rebounds. A schematic of the test method is shown in Figure 9-12.

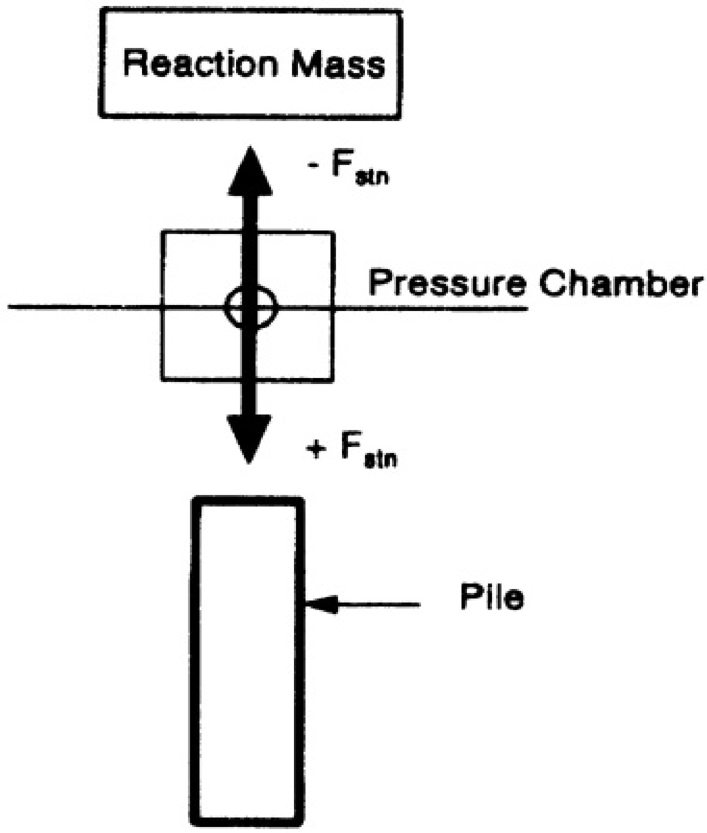


Figure 9-12: Schematic of Statnamic Test Method

Built-in instrumentation (load cell, accelerometers, and laser sensors) is used to measure load, acceleration and displacement. The instruments produce data that permit plots of load and displacement with time to be done in the field. The data is then converted into the familiar plot of load versus displacement shown in Figure 9-13 to permit interpretation of the failure load.

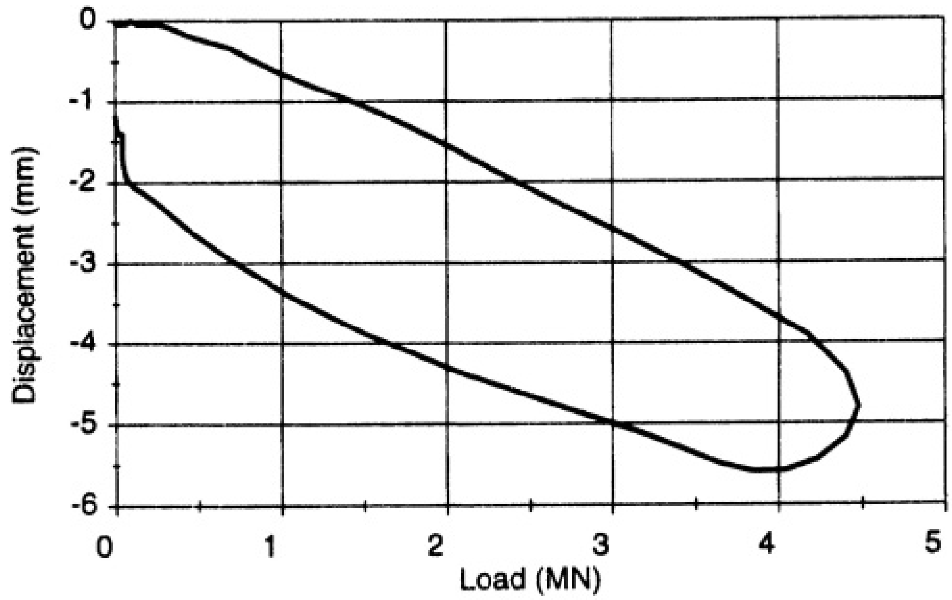


Figure 9-13: Load vs. Displacement plot generated from Statnamic Test

9.15 APPLE FREEWAY DESIGN EXAMPLE – WAVE EQUATION ANALYSIS

In this chapter the Apple Freeway Design Example is use to illustrate the wave equation analysis using the GRLWEAP program. The use of GRLWEAP for pile driveability analysis, checking suitability of contractors driving system, and determining pile driving criteria is addressed.

Site Exploration	Terrain Reconnaissance Site Inspection Subsurface Borings	
Basic Soil Properties	Visual Description Classification Tests Soil Profile	
Laboratory Testing	P_o Diagram Test Request Consolidation Results Strength Results	
Slope Stability	Design Soil Profile Circular Arc Analysis Sliding Block Analysis Lateral Squeeze	
Embankment Settlement	Design Soil Profile Settlement Time – Rate Surcharge Vertical Drains	
Spread Footing Design	Design Soil Profile Pier Bearing Capacity Pier Settlement Abutment Settlement Vertical Drains Surface	
Pile Design	Design Soil Profile Static Analysis – Pier Pipe Pile H – Pile Static Analysis – abutment Pipe Pile H – Pile Driving Resistance Abutment Lateral Movement	
	Construction Monitoring	Wave Equation Hammer Approval Embankment Instrumentation

Apple Freeway Design Example – Construction Monitoring
Exhibit A.

APPLE FREEWAY WAVE EQUATION ANALYSIS

Given: Using the soil profile and pile driving resistance previously computed (Chapter 8)

Required: Complete wave equation analyses using the GRLWEAP program for the following:

- Driveability of the proposed design pile section
- Acceptance of contractors driving system
- Production pile driving criteria

Solution:

Driveability of Proposed Design Pile Section

Proposed pile section is a 12 $\times$ 84 H – Pile. The maximum driving resistance determined from static analyses is at the east abutment where the total driving resistance including embankment penetration is 689.4 kips. Perform wave equation analysis for the proposed pile section using the maximum driving resistances.

Step 1: Prepare Wave Equation Input:

1. Select hammer (IHAMR)

    - Hammer size selected from Figure 9-7 using maximum driving resistance of 689.4 kips (345 tons). Minimum hammer energy = 57,000 ft-lbs.

    - Using GRLWEAP help screen, scan hammer library for hammers with sufficient energies. Select Delmag 30 – 13 hammer which has slightly more energy than required (66,000 ft-lbs) to insure efficient driving.

2. Select uniform or non-uniform pile cross-section along the entire pile (NCROSS)

    - Select non-uniform option (1) because pile point will be used. Pile cross-section described on input screen NCROSS =1 as shown on "page 7" of the GRLINP input printout.

3. Select percent pile skin friction (IPERC)

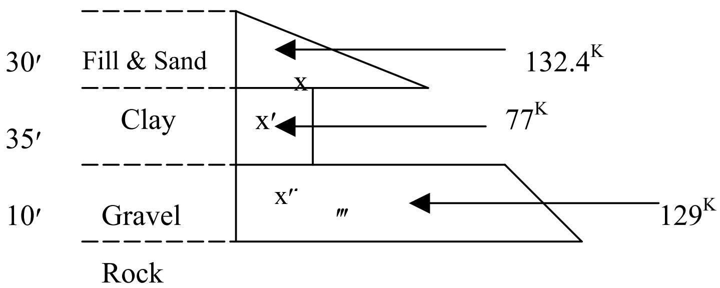
    - From static analysis, (skin friction resistance/total driving resistance) = 338/689 = 49%.

4. Select skin friction distribution (ITYS)

    - Use the actual skin friction distribution determined in the static analysis ultimate driving resistance computation (ITYS = 0). An analysis using the actual skin friction distribution is more realistic and accurate. From static analyses (see next page).

5. Select helmet and hammer cushion information (helmet weight, hammer cushion area, elastic modulus and thickness).
    - Using GRLWEAP help screen, scan hammer library for Delmag Hammers using the proposed

pile section.



$$x = \frac{132.4}{30} \times 2 = 8.83$$

$$x' = \frac{77}{35} = 2.2$$

$$x'' = \frac{5250}{5600}\left(\frac{129}{10}\right) = 12.08$$

$$x''' = \frac{5950}{5600}\left(\frac{129}{10}\right) = 13.72$$

6. Select Pile Top Information (length, x-section area, elastic modulus, specific weight, coefficient of restitution).

   - Using GRLWEAP help screen, obtain the x-sectional area and weight for the proposed HP12×84.

7. Select soil parameters (quake and damping).

   - Using GRLWEAP help screen, select appropriate soil parameters.

8. Select ultimate driving capacities to be analyzed.

   - Input a range of ultimate driving capacities around and including the maximum driving resistance calculated from static analyses (689.4 kips). A range of capacities highlights trends within the graphical plots.

GRLINP Input Screens for HP12×84 Driven through the Embankment Material – Ultimate Resistance 689.4 kips.

```
Title:  APPLE FREEWAY H-PILE @ ABUTMENT                          Page:    1

ANALYSIS OPTIONS
     IOUT       IJJ      IHAMR      IOSTR      IFUEL       IPEL
       .          .        13.          .          .          .

ANALYSIS OPTIONS
        N      ISPL     NCROSS     IBEDAM     IPERCS     ISMITH     DMPEXP
        .         .         1.          .        49.          .         .0

ANALYSIS OPTIONS
     ITYS      IPHI      IRSAO       ITER      IDAHA      IMAXT
       .         .          .          .          .          .

HELMET AND HAMMER CUSHION INFORMATION
   Helmet                         Hammer Cushion
   Weight       Area   ElasMod Thickness     C.O.R.   RoundOut Stiffness
     2.15     283.50     280.0     2.000       .800      .0100        .0
```

Title: APPLE FREEWAY H-PILE @ ABUT FULL EMBEDMT   Page: 2

PILE CUSHION INFORMATION

Area	Elastic Modulus	Thickness	C.o.R.	Round Out	Stiffness
.00	.0	.000	.500	.0100	.0

PILE TOP INFORMATION

Total Length	X-Sectn Area	Elastic Modulus	Specific Weight	C.o.R.	Round Out	D.O.A.1 Slck P2	D.O.A.2 Stiff P2
75.00	24.60	30000.0	492.00	.850	.0100	.00	.00

HAMMER OVERRIDE VALUES

Stroke	Effcy	Pressure	Reaction Weight	ComDelay Ign Vol	Comb Exp Coeff	Stroke Conv Crit
.00	.000	.0	.000	.000	.00	.00

SOIL PARAMETERS

Quake		Damping		Toe No. 2			
Skin	Toe	Skin	Toe	Quake	Damping	Fraction	Depth
.100	.100	.050	.150	.000	.000	.000	.00

Title: APPLE FREEWAY H-PILE @ ABUTMENT   Page: 3

ULTIMATE CAPACITIES
Give up to 10 Capacities (5 on first line)

350.00 550.00 631.80 689.40 740.00

ULTIMATE CAPACITIES (Continued)
(5 on the second line)

.00 .00 .00 .00 .00

CONTINUE GRLINP INPUT SCREENS FOR HP12x84 (689.4 kips).

Title: APPLE FREEWAY H-PILE @ ABUTMENT   Page: 7

NCROSS=1: NON-UNIFORM PROFILE, 1ST PILE   ** PILE LENGTH: 75.00

Depth	Area	Elastic Modulus	Specific Weight
.00	24.60	30000.0	492.00
74.60	24.60	30000.0	492.00
74.60	42.50	30000.0	492.00
75.00	42.50	30000.0	492.00

75500 PILE POINT (12" H)
AREA = 42.5 in^2
HEIGHT = 4.8 in

Title: APPLE FREEWAY H-PILE @ ABUTMENT Page: 8

ITYS=-1, 0: SKIN FRICTION DISTRIBUTION, 1ST PILE ** PILE LENGTH: 75.00

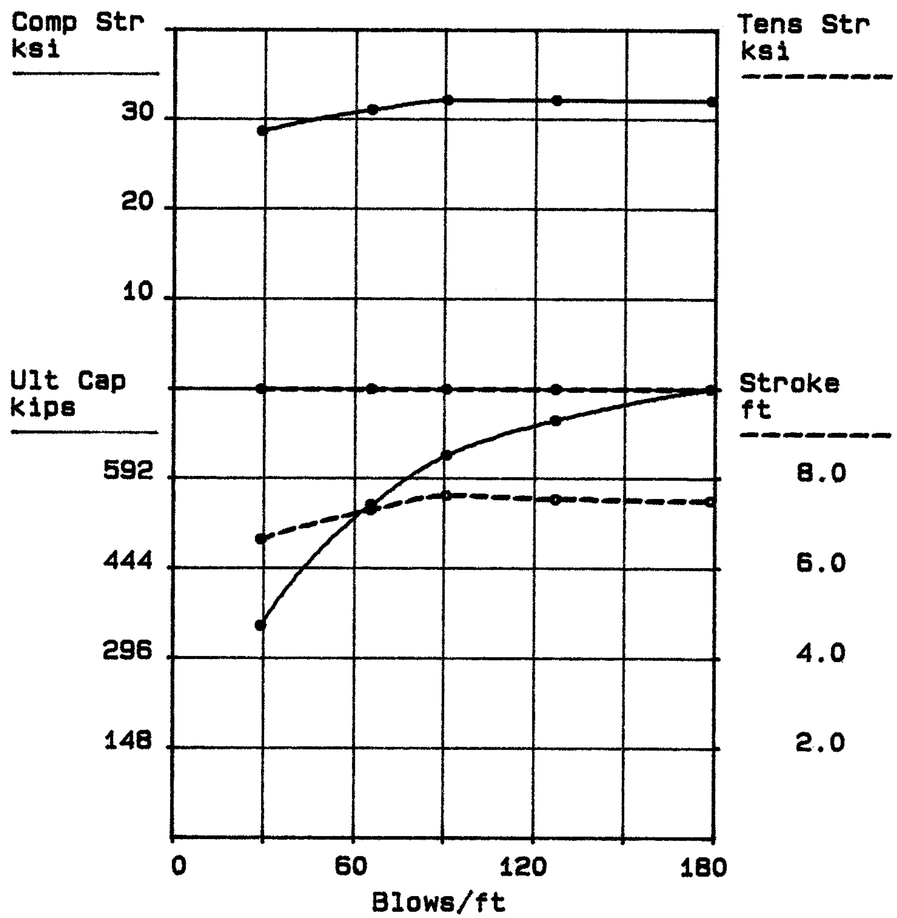
Depth	Relative Distribn
.00	.000
30.00	8.830
30.00	2.200
65.00	2.200
65.00	12.080
75.00	13.720

SUMMARY OF GRLWEAP RESULTS FOR HP12X84 (DELMAG 30-13)

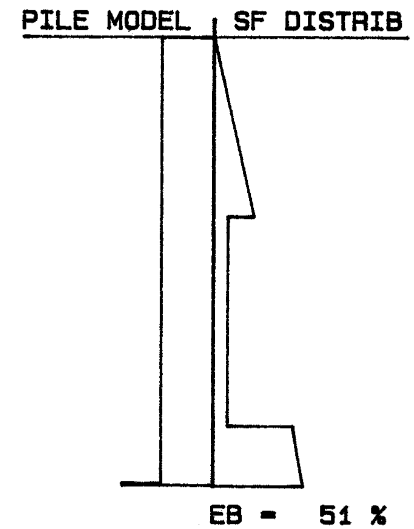
Rut (kips)	Bl Ct (bpf)	Stroke (ft) down	up	min Str (ksi)	max Str (ksi)	ENTHRU (kip-ft)	Bl Rt (b/min)
350.0	32.9	6.83	6.70	.00(	29.09(	25.9	45.3
550.0	79.6	7.38	7.32	.00(	31.53(	26.3	43.5
631.8	114.8	7.66	7.44	.00(	32.67(	26.9	42.9
689.4	170.6	7.51	7.47	.00(	32.46(	26.3	43.1
740.0	256.7	7.44	7.49	.00(	32.19(	25.9	43.2

G R L W E A P - Federal Highway Adm.

APPLE FREEWAY H-PILE @ ABUTMENT 05/10/93



DELMAG	D 30-13		
Efficiency		.720	
Helmet		2.15	kips
H Cushion		39690	k/in
Q =	.100	.100	in
J =	.050	.100	s/ft
Pile Length		75.00	ft
P-Top Area		24.60	in2



APPLE FREEWAY - HP12x84 DRIVEN THROUGH EMBANKMENT MATERIAL

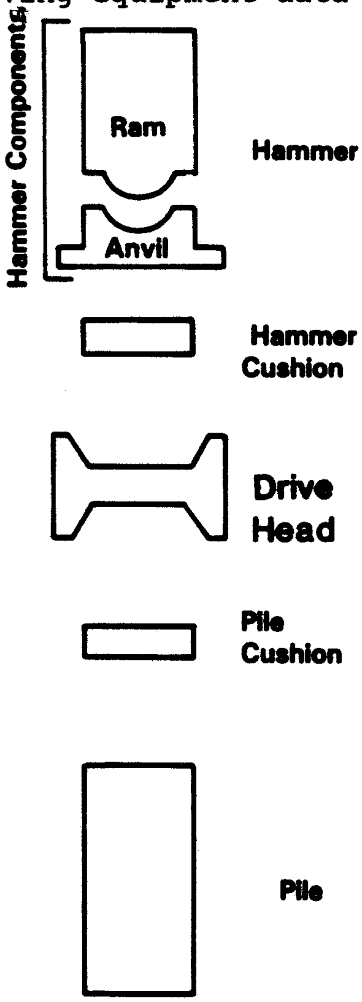
SUMMARY OF DRIVEABILITY ANALYSES FOR HP12X84

Pile section is adequate for the hardest driving conditions encountered at the east abutment. Driving stresses are below the maximum allowable driving stress of 32.4 ksi. Stresses are insensitive to the blow count and therefore, the pile won't be damaged when seated into the rock.

HP12X84 pile section is acceptable ✓

ACCEPTANCE OF CONTRACTORS DRIVING SYSTEM

Contractor has submitted a ICE 70-S driving system. The pile and driving equipment data sheet is shown below.



Hammer

Manufacturer: ICE Model: 70S
Type: OED Serial No.: 123
Rated Energy: 70,000 lb-ft at 10 ft Length of Stroke
Modifications:

Hammer Cushion

Material: Micarta
Thickness 2 in Area: 398 in²
Modulus of Elasticity – E 280,000 psi (P.S.I.)
Coefficient of Restitution-e 0.8

Drive Head – Helmet / Bonnet / Anvil Block / Pile Cap – Weight: 2.44 kips

Pile Cushion

Cushion Material: N/A
Thickness: Area:
Modulus of Elasticity – E (P.S.I.)
Coefficient of Restitution

Pile

Pile Type: HP12X84
Length (In-Leads): 75'
Weight/ft. 492 lb/ft³
Wall Thickness: Taper: No
Cross Sectional Area 24.6 in² in²
* Design Pile Capacity: 120 (Tons)
Description of Splice:

Tip Treatment Description:

* Driving Resistance: 280.5 tons @ Pier
344.7 tons @ Embankments

Perform wave equation analysis for the submitted driving system.

- Modify hammer data in the GRLWEAP input file previously used to analyze the HP12×84 pile section. Use the submitted driving system data, and the GRLWEAP driveability option.

GRLINP Input Screens for GRLWEAP Driveability option. Screens not Shown Below are Unchanged from the Pile Section Analysis.

```
Title:  FULL EMBEDMT - DRIVABILITY OPTION                        Page:    1

ANALYSIS OPTIONS
     IOUT       IJJ      IHAMR     IOSTR     IFUEL      IPEL
    -100.        .        129.       .         .          .

ANALYSIS OPTIONS
        N       ISPL     NCROSS    IBEDAM    IPERCS    ISMITH    DMPEXP
        .        .         1.        .         .          .         .0

ANALYSIS OPTIONS
     ITYS       IPHI     IRSAO      ITER     IDAHA      IMAXT
        .        .         .         .         .          .

HELMET AND HAMMER CUSHION INFORMATION
   Helmet                         Hammer Cushion
   Weight      Area   ElasMod Thickness    C.O.R.  RoundOut Stiffness
     2.44    398.00     280.0     2.000      .800     .0100        .0
```

Continue GRLINP Input Screens for GRLWEAP Driveability Options Analysis of Ice 70s Driving System.

```
Title:  FULL EMBEDMT - DRIVABILITY OPTION                        Page:    3

IPERCS = 0:  Friction Loss/Gain Factors
Give up to 10 Friction Loss/Gain Factors (5 on first line)
      .90      1.00      1.10       .00       .00

IPERCS = 0:  Friction Loss/Gain Factors
(5 on the second line)
      .00       .00       .00       .00       .00
```

Title: FULL EMBEDMT - DRIVABILITY OPTION Page: 6

IPERCS=0: DEPTHS & DRIVING SYSTEMS MODIFICATIONS ** PILE LENGTH: 75.00

Analysis Depth	Stroke	IFUEL	Efficiency	Stiffn. Factor	Cushion CoR
20.00	.00	.0	.00	.00	.00
30.00	.00	.0	.00	.00	.00
45.00	.00	.0	.00	.00	.00
65.00	.00	.0	.00	.00	.00
74.00	.00	.0	.00	.00	.00
74.50	.00	.0	.00	.00	.00
74.88	.00	.0	.00	.00	.00
75.00	.00	.0	.00	.00	.00
.00	.00	.0	.00	.00	.00

Title: FULL EMBEDMT - DRIVABILITY OPTION Page: 8

IPERCS = 0: SOIL PARAMETERS VS DEPTH ** PILE LENGTH: 75.00

Depth	Skin Friction	End Bearing	Skin Quake	Toe Quake	Skin Damping	Toe Damping	Sens.
.00	.000	5.000	.100	.100	.050	.150	.000
30.00	8.830	58.000	.100	.100	.050	.150	.000
30.00	2.200	5.000	.100	.100	.200	.150	.000
65.00	2.200	5.000	.100	.100	.200	.150	.000
65.00	12.080	96.000	.100	.100	.050	.150	.000
75.00	13.720	351.000	.100	.100	.050	.150	.000

SUMMARY RESULTS OVER DEPTH FOR ICE 70S HAMMER

FRICTION LOSS/GAIN FACTOR: .900

Depth (ft)	Rut (kips)	Frictn (kips)	End Bg (kips)	Bl Ct (bpf)	max Str (ksi)	min Str (ksi)	Bl Rte (b/min)	ENTHRU (kip-ft)
20.0	83.7	48.4	35.3	4.1	15.493	.000	45.4	34.8
30.0	170.1	114.4	55.7	8.5	25.056	-.032	40.8	35.0
45.0	148.1	143.3	4.8	7.2	23.653	-4.327	41.7	33.6
65.0	189.3	184.5	4.9	10.9	27.935	-7.017	39.9	31.3
74.0	*384.7*	*289.6*	*95.1*	*31.3*	*31.664*	*-.150*	*37.2*	*29.3*
74.5	*390.8*	*295.7*	*95.1*	*31.6*	*31.888*	*-.148*	*37.1*	*29.7*
74.9	*395.5*	*300.4*	*95.1*	*32.1*	*31.942*	*-.154*	*37.1*	*29.7*
75.0	*651.9*	*302.9*	*349.0*	*128.3*	*31.664*	*.000*	*36.8*	*30.0*

Total Driving Time 15.72 min; Total No. of Blows 631

FRICTION LOSS/GAIN FACTOR: 1.000

Depth (ft)	Rut (kips)	Frictn (kips)	End Bg (kips)	Bl Ct (bpf)	max Str (ksi)	min Str (ksi)	Bl Rte (b/min)	ENTHRU (kip-ft)
20.0	89.6	54.1	35.5	4.3	15.098	.000	45.3	34.0
30.0	183.3	127.5	55.8	9.7	24.582	-.135	40.9	32.0
45.0	164.6	159.8	4.8	8.4	24.303	-4.221	41.2	32.5
65.0	210.3	205.4	4.9	12.8	28.075	-6.177	39.5	30.2
74.0	*417.1*	*322.0*	*95.1*	*35.1*	*32.602*	*-.115*	*37.0*	*29.5*
74.5	*423.9*	*328.8*	*95.2*	*36.1*	*32.693*	*-.069*	*37.0*	*29.4*
74.9	*429.1*	*334.0*	*95.2*	*36.8*	*32.740*	*-.052*	*37.0*	*29.4*
75.0	*685.8*	*336.7*	*349.1*	*158.6*	*32.103*	*.000*	*37.3*	*29.6*

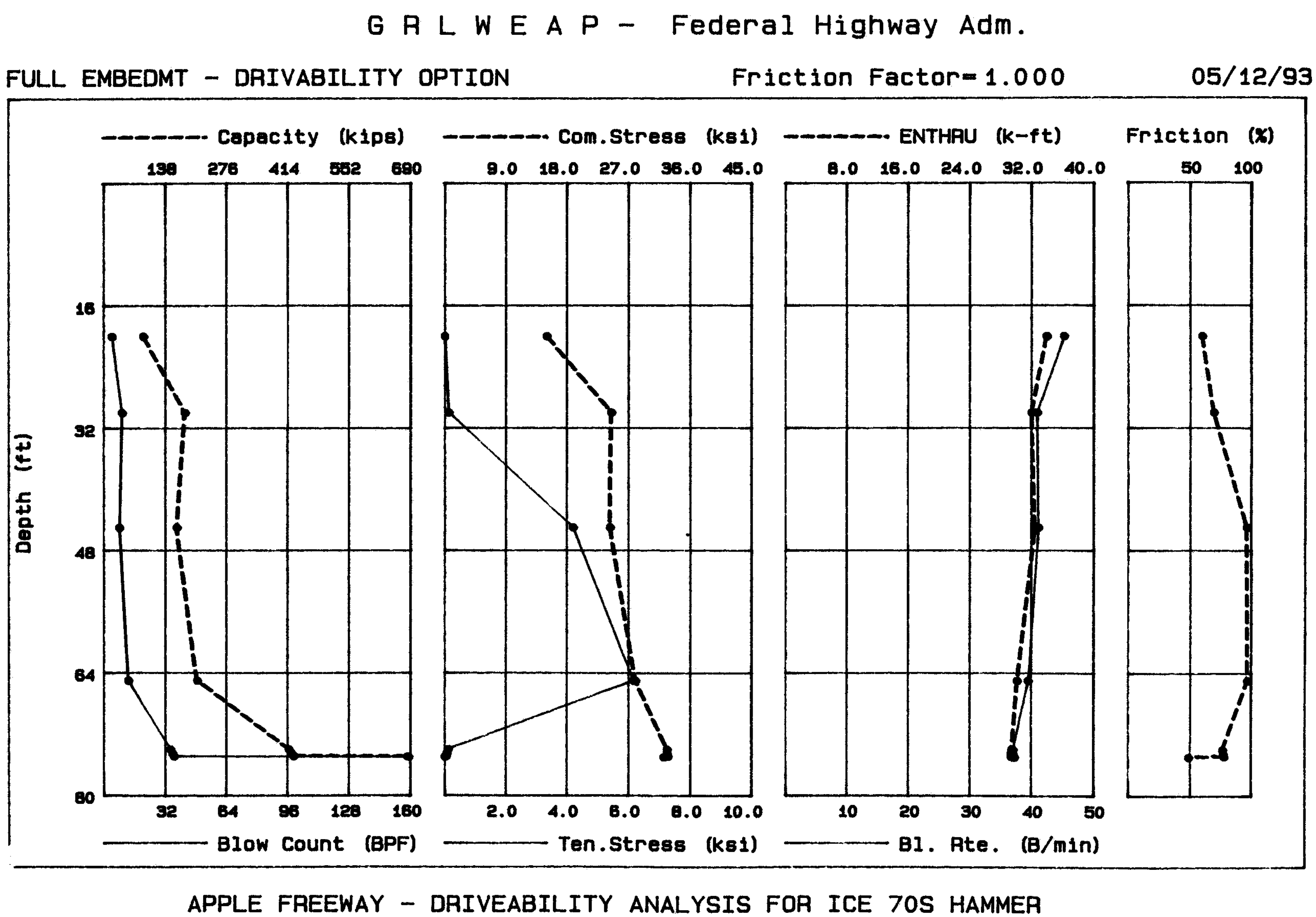
Total Driving Time 18.08 min; Total No. of Blows 720

FRICTION LOSS/GAIN FACTOR: 1.100

Depth (ft)	Rut (kips)	Frictn (kips)	End Bg (kips)	Bl Ct (bpf)	max Str (ksi)	min Str (ksi)	Bl Rte (b/min)	ENTHRU (kip-ft)
20.0	95.5	59.8	35.7	4.5	16.319	.000	44.9	34.6
30.0	196.6	140.6	56.0	10.2	25.761	-.978	40.6	32.9
45.0	181.2	176.3	4.8	9.3	26.005	-4.863	40.7	32.7
65.0	231.2	226.3	4.9	14.3	29.119	-5.837	39.0	30.6
74.0	*449.6*	*354.4*	*95.2*	*40.4*	*33.106*	*-.338*	*37.4*	*29.2*
74.5	*457.1*	*361.9*	*95.2*	*41.6*	*33.171*	*-.348*	*37.4*	*29.2*
74.9	*462.8*	*367.6*	*95.2*	*42.5*	*33.191*	*-.350*	*37.4*	*29.1*
75.0	*719.6*	*370.5*	*349.2*	*204.9*	*32.518*	*-.066*	*37.3*	*29.2*

Total Driving Time 20.19 min; Total No. of Blows 799

GRLWEAP Output – plot



SUMMARY OF DRIVEABILITY ANALYSES FOR ICE 70S

Driving Stresses: 32.7 ≈ 32.4 ksi ✓ OKAY

Driving stresses vary between 31.66 ksi (0.9 friction reduction) to 33.19 ksi (1.1 friction reduction), well below the yield strength of 36 ksi. Since, the maximum stresses occur when the pile has penetrated the rock and is at near refusal conditions, the piles should be capable of being seated into the rock without damage.

Blow Count: 159 bpf ≈ 144 bpf ✓ OKAY

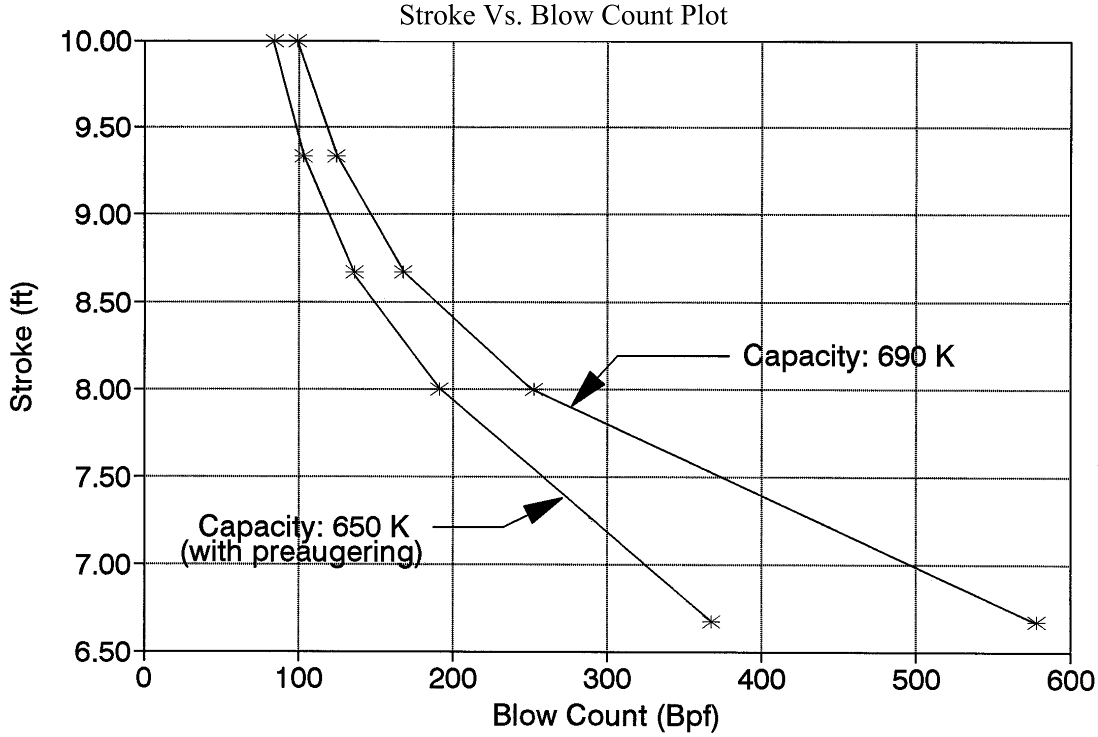
The blow count is approximately 35 bpf at just above the rock line, and near refusal 150 - 220 bpf in the rock layer. Therefore, the hammer should (if operating properly) penetrate quickly through the embankment and into the rock.

HAMMER APPROVED ✓

PRODUCTION PILE DRIVING CRITERIA FOR ICE 70S DRIVING SYSTEM

Drive HP12x84 pile through the embankment material and into the rock. Pile driving shall be terminated when the combination of stroke and blow count indicates a driving capacity of 690 kips. If preaugering is used the driving capacity of 650 kips should be attained.

Stroke Vs. Blow Count Plot



CONSTRUCTION CONTROL

- Pile Driveability

  Driveability of 12 x 84 H-Pile

  Section verified for most difficult driving condition.

- Driveability versus Depth

  Driveability of 12 x 84 computed for full 75' depth.

  Pile installation time expected to vary between 16 and 20 minutes (no preaugering).